SETTLEMENT PREDICTIONS BY STANDARDIZED METHODS

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by

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ABSTRACT

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The purpose of this thesis is to investigate the feasibility of using standardized methods of predicting total settlement and time rate of settlement and to evaluate the role of engineering judgement in these predictions.

Several standardized methods have been presented and standardized procedures for selecting parameters have been recommended. Settlement was predicted for seven field cases using the various methods presented. The resulting predictions were then compared with actual measured performance. A valid comparison between observed and predicted long term settlement was found impossible unless the average degree of consolidation was known. A comparison of initial settlements was feasible. Results indicated that standardized methods are feasible, but that standardized procedures for selecting parameters are not. Engineering judgement is required in the selection of the appropriate parameters to match the specified method. The results also showed the need for sufficient representative data and the need for a reasonable method of comparison between observed and predicted performance.

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UNCOMMON ABBREVIATIONS AND SYMBOLS

ε_{vn}	Strain due to virgin compression
ε_{vo}	Strain due to recompression
c_{vn}	Coefficient of consolidation in the normal Consolidated region
c_{vn}	Coefficient of consolidation in the over-consolidated region
c_v'	Coefficient of consolidation intermediate between c_{vn} and c_{vo}
1-D	One-Dimensional Method
Mod 1-D	Modified One-Dimensional Method
S-B	Skempton-Bjerrum Method
S-B(L.Y.)	Skempton-Bjerrum Method considering First Local Yield
E	Elastic Method
E(L.Y.)	Elastic Method considering First Local Yield

CHAPTER I

INTRODUCTION

1.1 PREDICTIONS

It is the job of the Civil Engineer to make predictions. A good prediction can not only save a client large sums of money, but may also help to prevent injury or loss of life due to failure. An engineer may be required to predict the:

1. Completion Date;
2. Construction Cost;
3. Factor of Safety;
4. Environmental Consequences;
5. Performance of the Structure.

As part of the performance predictions, a geotechnical engineer may have to forecast the magnitude, direction and rate of deformation of the structure on its foundation; the stability of the structure on its foundation; and the loads, both lateral and vertical, applied to structures resting on or within the soil (Lambe, 1973a). In order to make a prediction in a soil-rock system, the engineer uses the following steps:

1. Determines the field situation and then simplifies;

2. Determines the mechanism(s);
3. Selects a method and parameters;
4. Manipulates the method and parameters to obtain a prediction;
5. Portrays the prediction (Lambe, 1973a)

Although all phases of the prediction process are important, probably the most crucial are steps 3 and 4. The relationship which exists between the method and parameters is vital in making a good prediction.

The selection and manipulation of method and parameters in a soil-rock system is complicated by many factors. Some of these are listed below:

1. One cannot see underground;
2. Soil-rock strata may vary widely both horizontally and vertically;
3. Soil is not an isotropic and elastic medium;
4. Soil properties are dependent on stress level, time and environment;
5. Soil may be very sensative to disturbance. (Lambe, 1973b)

It is easy to understand, therefore, the difficulties the

geotechnical engineer may experience in making predictions. Nevertheless, because of the consequences which may result from poor predictions, those difficulties must be overcome.

In 1973, T.W. Lambe classified predictions according to the time when they are made and the information known at the time. This classification is shown below:

Prediction Type	When Prediction Made	Results at Time Prediction Made
A	Before Event	----
B	During Event	Not Known
B 1	During Event	Known
C	After Event	Not Known
C 1	After Event	Known

Type A predictions are made prior to construction using only data available at the time. These forecasts are the challenge of the geotechnical engineering profession. Type B predictions may be based on some information obtained during the initial phases of construction. They are slightly less valuable than Type A predictions and somewhat easier to make. Class C predictions are "autopsies", being made after the event. Prediction Types B 1 and C 1 are certainly easier to make than the others and are of less importance. Knowing the results before hand may either consciously or subconsciously effect the

engineer's prediction procedure. Type C 1 predictions aided by an experienced eraser often creates a "pseudo engineering judgement" factor which yields amazingly accurate results.

1.2 OBJECTIVE

The objective of this thesis is to investigate the feasibility of using standardized methods to predict settlement. By making uniform the manipulation of method with parameters, the author has attempted to standardize the engineering judgement used in each analysis. In order to evaluate these techniques, Type C predictions of settlement were made for seven case studies and then compared with actual performance measurements. Because Type C predictions were made, the possibility of any "pseudo engineering judgement" was eliminated. A description of each one of the field cases is given in the next section.

As a collateral goal, the author has tried to evaluate the role of engineering judgement in geotechnical predictions. If engineering judgement is required in soil-rock predictions, at what stage should it be used and should it be used in conjunction with a standardized method?

1.3 DESCRIPTION OF CASE STUDIED

1.3.1 Alibey Test Embankment

The Alibey embankment was built as part of the Alibey Reservoir Project near Istanbul, Turkey. Initially built in 1967 and 1968 to evaluate performance predictions, the embankment was designed as a cofferdam to divert the flow of the Alibey Creek during the construction of the main embankment of the earth dam. The site is in a wide valley underlain by alluvium consisting of about 30 meters of soft plastic clay with occasional sand lenses resting on a base of stiff clays and dense sands. The test embankment was constructed to a height of about 7.5 meters with the base being about 81 meters wide and with sides sloping at about 1 on 3. There are two clusters of instruments which include 10 hydraulic piezometers, 12 vibrating wire piezometers, and 2 settlement plates (Dugan 1968). The embankment and subsurface profile are shown in Chapter 3.

1.3.2 Interstate Route 95 (Station 233+50)

This section of Interstate Route 95 was constructed in 1968 and 1969 and is located in the town of Saugus, Massachusetts. The 35 foot high embankment is built over a low tidal swamp with a mean ground elevation of +5 feet and with maximum high tides at about elevation +10 feet. The subsoil consists of a layer of peat, one of fine sand to silty sand, and then about 140 feet of soft to very soft gray clay which

is underlain by glacial till and bedrock. The gray clay is overconsolidated by dessication to a depth of about 80 feet. The layer of peat directly under the 217 foot embankment base has been removed and replaced with the granular embankment material. The embankment sides slope at approximately 1 on 2 (Storch Engineers, 1965). The instrumentation consists of 1 settlement plate, 3 hydraulic piezometers, and 4 slope indicators. The soil profile and embankment cross section are shown in Chapter III.

1.3.3 Interstate Route 95 (Station 263+00)

This section of Route I-95 is similar to that located at Station 233+50 with the following exceptions: the embankment rises 40 feet above ground surface and is 245 feet at the base; the stratum of soft to very soft clay is 90 feet thick (Storch Engineers, 1965). The instrumentation is similar to that at Station 233+50.

1.3.4 Interstate Route 95 (Station 276+00)

Again, this section of Route I-95 is similar to that at Station 233+50 with the following exceptions: the clay layer is only about 54 feet thick, the embankment was constructed to about 20 feet above the ground surface; and it is 260 feet wide at the base (Storch Engineers, 1965).

The instrumentation consists of 1 settlement plate and 2 hydraulic piezometers.

1.3.5 M.I.T. Test Section

This highly instrumented section of embankment is located at Station 246+00 of Interstate Route 95, Saugus, Massachusetts. The general geological features are similar to those described in Section 1.3.2. The embankment was constructed to a height of about 35 feet above the ground surface with a base dimension of about 240 feet. The sides slope at approximately 1 on 2. The clay stratum, which progresses in consistency with depth from firm to soft to very soft, is about 136 feet thick (Guertin, 1962). The instrumentation consists of the following: 33 hydraulic piezometers, 6 vibrating wire piezometers, 5 observation wells, 12 settlement rods, 5 settlement platforms, 6 inclinometers, 1 benchmark, and 9 total stress cells (Wolfskill and Soydemire, 1971).

1.3.6 Northeast Test Embankment

The Northeast Test Embankment, constructed in 1957, was designed as part of the Revere Beach connector to Interstate Route 95 in the town of Revere, Massachusetts. The embankment section is approximately 60 feet long by 200 feet

side and rises about 35 feet above the existing ground surface. The subsurface conditions are similar to those previously described for the sections of Route I-95 with the following exception: the tope 8 feet of the clay stratum is hard while the lower 91 feet of clay progresses in consistency from medium to soft. The section is instrumented with 3 settlement platforms, 44 surface displacement stakes, 4 hydraulic piezometers, and 2 settlement anchors (Karlsrud, 1968).

1.3.7 Lagunillas Preload

The Lagunillas Preload project is a series of circular embankments designed to increase the undrained shear strength of the foundations of three heavily loaded tanks. Previous investigations indicated that if the tanks were constructed and filled without preloading, failure would result. The subsoil conditions which exist at Lagunillas, on the east coast of Lake Maracaibo, Venezuela, consist of a stratum of fine silty sand which grades with depth to a sandy silt and finally to a soft clay. Below the clay layer lies dense sands and stiff clays. The particular preload studied here is for a tank with a diameter of 50 feet and a height of 56 feet. The embankment itself rises about 35 feet above ground surface, has a base diameter of 138 feet, and has sides which slope at about 3 on 4. Instrumentation consists of 7

piezometers and 5 settlement plates. The embankment and subsurface profile are shown in Chapter 3.

CHAPTER II

METHOD OF SETTLEMENT ANALYSIS

2.1 GENERAL

The procedure used to make settlement predictions is similar to stated in Chapter I. First, the subsoil profile had to be determined as accurately as possible using the results of borings. Next these results were simplified into an accurate model of the subsurface situation. Then the load shape and loading time period were simplified and modelled. The exact standardized procedure used in modelling is explained in further detail in Appendix A.

After the idealization of the field situation, the mechanism was determined. A mechanism is closely related to the method of analysis because the method, in theory, is a numerical model of the actual field mechanism. In each of the seven case studies presented here, various methods were used and therefore, different mechanisms were considered. An example of this is the Lagunillas preload case study in which a circular load was placed on soft clay. The situation is truly not one-dimensional compression, however, one method of analysis used was the One-Dimensional Method. Although a method may not be an exact theoretical representation of the

actual mechanism, it may yield a correct prediction when combined with a particular set of parameters. Additionally, there may be no simple method which describes the real field situation and an inexact method may have to be used.

After a method was chosen, parameters were determined from laboratory or field tests. Because the method and data are often incomplete and inaccurate representations of actual situations, and because nearly all soil engineering problems are statically indeterminate, methods and data must be manipulated to produce a reasonable prediction (Lambe, 1973a). The methods and paramters used to predict settlement for the seven cases studied are described briefly in the following sections. More detailed, step-by-step techniques are listed in Appendix B.

2.2 METHODS AND PARAMETERS USED TO PREDICTE TOTAL SETTLEMENT

2.2.1 One-Dimensional Method

This method assumes that the mechanism which causes settlement is one-dimensional strain. It also assumes that the change in vertical stress is constant with depth and that no undrained deformation (initial settlement) occurs within the cohesive strata. Initial settlement due to loose cohesionless layers is computed, however. The instances in which one-dimensional strain occurs may be rare. Nevertheless,

when the loaded area becomes very large and the thickness of the compressible layers small, the strains under the center of the load are then closely approximated by this mechanism.

The parameters required for this type of analysis are obtained from the void ratio versus logarithm stress (e-log $\bar{p}$) curves of oedometer tests on samples from various depths throughout the cohesive layers. Schmertmann's method for overcoming the effects of sample distrubance should be used to determine the maximum past pressure ($\bar{\sigma}_{vm}$) and to construct the "field" e-log $\bar{p}$ curve (Ladd, 1971). The values for the Compression Ration (CR) and the Recompression Ratio (RR) are taken from the field curve and a weighted average of each is found over the depth. The initial effective vertical stress ($\bar{\sigma}_{vo}$) should be determined by the procedure set forth by Lambe and Whitman (1969). The final vertical effective stress plus the applied load.

The One-Dimensional Method requires that compressible strata be divided into layers and settlements be computed for each layer. Recommendations for layering of the subsoil are located in Appendix A and the computational techniques in Appendix B.

2.2.2 Modified One-Dimensional Method

This method is very similar to the one previously described. The only major difference being that the change in vertical stress ($\Delta\sigma_v$) is not constant with depth and thus the final vertical effective stress ($\bar{\sigma}_{vf}$) differs from that of the One-Dimensional Method. The change in vertical effective stress with depth should be computed from elastic theory where Poisson's Ratio is equal to one-half (μ_i=1/2). Charts of elastic solutions have been published by Lysmer and Duncan (1972), Davis and Poulos (1973), Lambe and Whitman (1969), and others. The final effective vertical stress is the sum of the initial vertical effective stress and the change in vertical stress.

2.2.3 Elastic Method

The Elastic Method assumes that a soil stratum is an isotropic elastic-medium which can be represented by a set of elastic constants. When the constants have been determined, predictions can be made with the aid of charts and tables which have been published (Lysmer and Duncan, 1972; Davis and Poulos, 1968). Total settlement is considered to be composed of two components, initial elastic settlement (ρ_e) and consolidation settlement (ρ_c), due to undrained and drained conditions resprectively. Both components are computed by the same method but different paramters. The parameters required for initial elastic settlement predictions are an undrained Young's Modulus (E_i) and an undrained Poisson's Ration (μ_i). The parameters required for computation of consolidation settlement are a drained modulus (E_c) and a drained value of Poisson's Ratio (μ_c). The procedure for determination of these parameters and preferences recommended in selecting the data are given in Appendix B. When values of Young's modulus vary with depth, a weighted average should be determined for the analysis.

The techniques used in computation of the Elastic Method are taken from a compilation of tables and charts of elastic solutions published by Lysmer and Duncan (1972). Other sources are available, but all Elastic Method

predictions made for the seven cases studied here were computed by means of the Lysmer and Duncan solutions.

2.2.4 Elastic Method Considering First Local Yield

This method, proposed by D'Appolonia, Poulos, and Ladd (1971), modifies the inital settlement predicted in Section 2.2.3 to account for yielding of highly stressed areas. When first local yield occurs, the stress-strain properties of that volume of soil become nonlinear and plastic flow occurs. As a result, the initial undrained deformation is no longer linearly elastic. This method utilizes graphs prepared from the results of nonlinear finite element analyses to give a factor, the Settlement Ratio (S_r), which can be applied to modify the initial eastic settlement (ρ_e) (D'Appolonia, et al., 1971). In order to use these graphs, the following parameters must be known:

1. The loading and subsurface geometry (H/B);
2. The ultimate bearing capacity (q_{ult}) which is a function of the undrained strength (s_u) and geometry;
3. The intial shear stress ration (f) which is a function of the initial

vertical and horizontal effective stresses ($\bar{\sigma}_{vo}$ and $\bar{\sigma}_{ho}$) and the undrained shear strength;

4. The inital elastic settlement (ρ_e) computed by the method described in Section 2.2.3.

2.2.5 Skempton-Bjerrum Method

The basic premise of this method is that consolidation settlements occur as a result of the dissipation of excess pore pressures (Δu) and that the excess pore pressures are not necessarily equal to the change in vertical stress caused by the applied load. Additionally, the Skempton-Bjerrum Method allows for initial settlements due to undrained, non-one-dimensional, deformation. Initial settlements are computed from either the purely elastic method (Section 2.2.3) or the method which considers the effect of first local yield (Section 2.2.4).

Since consolidation settlement is a function of the excess pore pressure, an important paramter required for this method is Skempton's A parameter. Also, because the dissipation of the pore pressure is what causes the settlement, the loading geometry and stratigraphy must be considered. Skempton and Bjerrum (1957) proposed a semi-empirical factor

(μ') to apply to the consolidation settlement calculated using the Modified One-Dimensional Method (Section 2.2.2). This factor is a function of Skempton's A parameter and the geometry. Computation of the μ' factor is explained in Appendix B.

2.3 TIME RATE OF SETTLEMENT

2.3.1 One-Dimensional Consolidation Method

This method assumes that Terzaghi's theory of consolidation is valid and that flow occurs in only one direction. The parameters required are:

1. coefficient of consolidation (c_v) determined by laboratory oedometer tests as described in Section 2.3.3 and Appendix C;
2. time factor (T) as related to the average degree of consolidation ($\bar{U}$) according to the Terzaghi theory listed in Appendix C (Hough, 1969).
3. drainage distance (H_d) from the sub-surface profile as discussed in Appendix C.

The amount of settlement at a certain time is related to the percent consolidation at that time as defined

by Terzaghi theory. The time is proportional to the time factor and the square of the drainage path and inversely proportional to the coefficient of consolidation. The techniques for predicting settlement versus time when the load is applied gradually is explained in Appendix C and is based on a method described by Taylor (1948).

2.3.2 Two-Dimensional Consolidation Method

This method is based on work done by Christian, Boehmer, and Martin (1972) and accounts for lateral drainage under a strip load. The difference between this method and the one previously described is in the determination of the time factor (T). Two-dimensional drainage causes the time factor to be effected by the geometry of the load and the stratigraphy. It is also effected by anisotropic permeabilities, but these effects were not considered in the seven cases studied here. As recommended by Christian, et al. (1972), the one-dimensional coefficient of consolidation defined by Terzaghi theory is Satisfactory for use with this method. The exact procedure for making predictions by the Two-Dimensional Consolidation Method is listed in Appendix C.

2.3.3 Selection of the Coefficient of Consolidation (c_v)

Because the coefficient of consolidation may vary drastically with stress range and depth, it is difficult to

assign one value to c_v for use in a time-settlement analysis. Ladd (1971) discusses the causes of the large variance of c_v. Younan and Barden (1969) list several approximate methods of treating the variance of c_v with depth. Among the methods are a finite difference technique , Terzaghi's method for averaging soil coefficients, a transformation method, and some cruder approximations. The method which was used in the seven case studies presented here is the transformation by Palmer (1953) in NavDocks,DM-7. With this method, the subsoil is divided into layers of approximately the same coefficient of consolidation. The heights of the layers are then progressively transformed to yield a final product of one equivalent layer with one value of c_v. The main assumption in this transformation is that the time factor (T) of two adjacent layers is the same. By equating the time factors, the height of one of the layers can be transformed to correspond to a common c_v for the two layers. This approximation is not theoretically correct because the continuity of flow at the interface between the two soil layers is not maintained. Nevertheless, because Palmer's transformation is simple to perform and gives reasonable results, this method was chosen for use in the seven field cases studied here (Barden and Younan, 1969).

The problem of the coefficient of consolidation varying throughout the stress range has not been studied in

as much detail as that of its variance with depth. J. E. Garlanger[1] recommended that when a soil is loaded from the overconsolidated range into the normally consolidated range, the coefficient of consolidation be evaluated by determining its basic components. In other words, first determine the coefficient of volume change (m_v) and the coefficient of permeability (k) and then calculate $c_v = \frac{k}{m_v \gamma_w}$.

This method was considered for use here, but the difficulty in evaluating the coefficient of permeability with the data available justified employing another means. Upon the recommendation of C. C. Ladd[2], a method was devised in which the value of c_v in the overconsolidated range and that in the normally consolidated range were compared to the strain due to recompression and that due to virgin compression. From this comparison one value of c_v was obtained. The detailed procedure for determining the value of the coefficient of consolidation is listed in Appendix C. A graphic method is also shown.

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CHAPTER III

SETTLEMENT PREDICTIONS

3.1 PARAMETERS AND RESULTS

Models of the seven (7) case studies and the parameters used in each method of analysis are presented in figures and tables located in this chapter. Figures 3-1 through 3-7 show the idealized load, the soil profile, and the stress history of the seven case studies. Table 3-1 summarizes some fundamental parameters used in each prediction as well as the reference sources for all data. Table 3-2 presents the results of all initial settlement and consolidation settlement predictions. The results of the time rate of settlement predictions, in the form of average degree of consolidation versus time, are listed in Table 3-3. Abbreviations used in the performance plots presented in this chapter are located in the "Uncommon Abbreviations and Symbols" list.

3.2 COMPARISON BETWEEN PREDICTED AND OBSERVED PERFORMANCE

3.2.1 General

This section of Cahpter 3 deals with the evaluation of the various methods of predicting settlement for

the seven cases studied. The comparison between predicted and observed is made graphically in Figures 3-8 through 3-35 by use of settlement versus time plots. Also included in this group of figures are plots of observed settlement versus the logarithm of time. Table 3-4 shows a comparison of the predictions and measurements of initial settlement. Observed settlement at the end of the loading period is compared with that predicted by the Skempton-Bjerrum, Elastic and Elastic-Local Yield Methods. The following sub-sections discuss the settlement comparison for each of the seven case studies.

3.2.2 Alibey Test Embankment

Figures 3-8 through 3-11 are performance plots for the Alibey cofferdam. Making a comparison between observed and predicted settlement is difficult because only limited measured performance data is available. The last value of measured settlement which is available was made about 70 days after construction was completed. Nevertheless, a comparison of early settlements should be valid. Figure 3-8 shows that early settlement is grossly under-predicted by all methods. Computation of initial settlement for the elastic and Skempton-Bjerrum Method is much too small. This difference may be due to the extremely high Young's modulus

which was obtained from Anisotropically (K_o) Consolidated Triaxial Compression Tests ($\overline{CK_oUC}$).

Figures 3-9 and 3-10 show the predicted long term settlements. A meaningful comparison between these and the actual performance is extremely difficult if not impossible, to make. The plot of the observed settlement versus the logarithm of time, Figure 3-11, seems to indicate that primary consolidation was still proceeding at the time of the last measurement and that this last measured settlement is not the total final settlement (ρ_{TF}). Because neither the final consolidation settlement nor the average degree of consolidation at the time of the last measurement is known, it is impossible to evaluate the prediction of total settlement.

The prediction of time rate of settlement are also extremely difficult to evaluate. Some of the sources of error may be the drainage distance (H_d), the coefficient of consolidation (c_v), the mechanism of consolidation itself, and the possibility of anisotropic permeabilities. Lambe (1973a) backfigured a value of c_v using field piezometer measurements. His results showed that c_v was 25 times that of the laboratory value. This combined with the existance of shale and silt layers may contribute to the large deviation between predicted and observed behavior. The vaiation noted in the predictions of the One-Dimensional and of the

Two-Dimension Consolidation Methods was very small compared to the difference between predicted and observed. A comparison of these two methods is not realisitc for this case study.

3.2.3 I-95 Station 233+50

Figures 3-12 through 3-15 represent settlement predictions and observations for the Interstate Route I-95 embankment at Station 233+50. Figure 3-12 shows the comparison between predicted and observed performance during the early stages of consolidation. It is obvious that the elastic and elastic-local yield methods of predicting initial settlement were too large. This is probably mainly due to the low value of Young's modulus which was obtained from Unconsolidated Undrained and Unconfined Compression tests.

Figures 3-13 and 3-14 show the comparison be predicted and observed performance over a long period of time. Nevertheless, because the embankment was constructed so recently, it is difficult to evaluate the long term settlement predictions. For the same reasons given inthe case of the Alibey Dam, it is also difficult to compare time rate of settlement predictions. Figure 3-15 is a plot of settlement versus log time. It seems to indicate that primary consolidation is still going on.

3.2.4 I-95 Station 263+00

Predicted and observed performance for Station 263-00 of Interstate Route 95 are portrayed in Figures 3-16 through 3-19. The early stages of settlement are shown in Figure 3-16. The predictions of initial settlement are larger than that which was measured. This is probably due to the very low value of Young's modulus which was obtained from UU tests.

For the reasons previously stated, the comparison of long term settlement and time rate of settlement is very difficult. It is obvious, however, that there is a large range of predicted values of total settlement. This large range is probably due to the low value of modulus used. The total settlement estimated by the Elastic Method is over four times that estimated by the One-Dimensional Method.

The actual settlement is plotted versus log time in Figure 3-19. As in the other cases previously examined, the clay stratum here also appears to be still undergoing primary compression.

3.2.5 I-95 Station 276+00

Figures 3-20 through 3-23 are performance plots for Station 276+00 of Interstate Route 95. Figure 3-20 compares predicted with observed settlement during and shortly after

construction. The prediction of initial settlement using the Skempton-Bjerrum method seems to be fairly close to the measured value at the end of construction. At first it may seem incongruous that an initial settlement prediction may be nearly accurate in this case but very inaccurate in two other similar situations. The reason may be partially due to the parameters which were used. At Station 276+00 Young's Modulus was derived from Isotropically Consolidated Triaxial Compression Tests ($\overline{CIU}C$) and some Unconsolidated Undrained Tests (UU). At Stations 233+50 and 263+00, the main source of Young's Modulus was only UU and U tests. Another reason for the apparent accuracy of the Skempton-Bjerrum prediction of settlement at the end of the construction period may be the low value of total consolidation settlement (ρ_{cf}) determined by that method. Therefore, even if the initial settlement prediction were a little high, the low value of consolidation settlement would compensate in the early phases of consolidation. Figures 3-21 and 3-22 indicate that this is what probably happened.

These figures also indicate that the One-Dimensional Method, the Modified One-Dimensional Method, and the Skempton-Bjerrum Method all under-predict the actual settlement. The casue of this is most likely the very low Compression Ratio and Recompression Ratio determined from oedometer tests.

(See Table 3-1 for comparison with other values obtained.) The Elastic Method seems to grealy over-predict the total settlement. This may be due to the empirical method used to get the drained modulus (E_c = 1/3 E_i).

By comparing the shapes of the observed and the predicted settlement, it appears that the estimates of time rate of settlement may be reasonable. This may be due to a better selection of parameters, a better fitting method, or just a fortuitous concellation of errors.

The difference between the One-Dimensional Consolidation Method and the Two-Dimensional Consolidation Method of making time rate predictions is only slightly in comparison with the differences in observed and predicted behavior.

3.2.6 M.I.T. Test Section

The comparison between observed and predicted performance of the M.I.T. Test Section is graphically portrayed in Figure 3-24 through 3-27. Figure 3-24 shows close agreement between the actual initial settlement and that predicted by the Elastic-Local Yield Method. The Young's modulus used in this prediction was obtained from Anisotropically (K_o) Consolidated Triaxial Compression Tests ($\overline{CK_oU}C$). The influence of the local yield was small since $\frac{q_s}{q_{ult}} = 0.48$. The Skempton-Bjerrum Method results in an initial settlement prediction fairly close to the actual value also. The accuracy in both of these cases may be due to the type of tests performed and the quality of samples obtained.

Figures 3-25 and 3-26 show the comparison of long term settlement predictions and measurements using the two different time rate of settlement methods. As was the case in the first few examples, it is very difficulty to evaluate this comparison. The time rate estimate may be accurate and the total settlement may be inaccurate or visa-versa. There are very many sources of error, and, therefore, it is difficult to evaluate the results without an accompanying pore pressure analysis. A graph of actual settlement versus the logarithm of time is plotted in Figure 3-27. It seems to indicate

that primary compression is still in progress.

3.2.7 Northeast Test Embankment

Figures 3-28 through 3-21 are settlement plots for the Northeast Test Embankment. The observed performance is very interesting in that three settlement platforms spaced only 100 feet apart, each settled quite differently. Nevertheless, subsoil exploration showed approximately the same stratigraphy under each platform. Lambe, et al. (1971) discuss the possible causes of this variance and conclude that the information available is not sufficient to explain the differences in observed behavior.

Figure 3-28 compares predicted with observed performance during the early stages of settlement. Although three distinct settlement curves are shown for the three settlement platforms, the variation at the end of the loading period is small. The actual settlements at the end of the loading period are very close to those predicted by all methods. Again, elastic methods used $\overline{CK_oUC}$ parameters in estimating initial settlement.

Figures 3-29 and 3-30 show long term predictions and measurements. The extremely large variation among the three settlement platforms is larger than the variation among the different prediction method. The One-Dimensional and

Modified One-Dimensional Method seem to accurately predict the amount of total settlement for settlement platform-PL-1, but this may be caused by a lucky cancellation of errors.

Figure 3-31 shows measured settlement versus log time. It appears that the curve for platform PL-1 is beginning to become more horizontal, indicating that primary consolidation may be nearing completion. Nevertheless, it does not seem to be ending under the two other platforms. More investigation is needed to explain the observed behavior.

3.2.8 Lagunillas Preload

Figures 3-22 through 3-25 are performance plots for the Lagunillas Preload. Figure 3-32 compares observed and predicted performance in the early stages of settlement. It is obvious that the methods used under predict the amount of settlement at the end of the loading period. The Young's modulus used in the elastic method was obtained mainly from $\overline{CIUC}$ tests. The method of estimating the influence of local yield indicates that the foundation would fail under the applied load. However, the embankment was constructed gradually and the foundation did not fail. It is felt that, although failure did not occur, the embankment and foundation were close to it. This may partially substantiate the fact that the

actual initial settlement was underpredicted by the elastic method.

Figures 3-33 and 3-24 show long term settlement predictions and measurements. For the same reasons stated in paragraph 3.2.2, a comparison between observed and predicted behavior is difficult. All methods underpredict the amount of total settlement. This may be due, at least in part, to the underestimate of initial settlement. Since the degree of consolidation was not predicted, it is difficult to determine the additional quantity of primary consolidation which will occur. Figure 3-35 does not emphatically indicate a reduction in the rate of primary compression.

By comparing the shapes of the observed and predicted time-settlement curves in Figures 3-33 and 3-35, it seems that the time rate estimate may be inaccurate. This may be caused by an erroneous drainage distance, by an incorrect coefficient of consolidation, by the existence of anisotropic permeabilities, or by a combination of any of these factors.

3.3 SUMMARY OF RESULTS

The comparison between observed and predicted initial settlements are sumarized in Table 3-4. The importance of the relationship between method and parameters

becomes apparent. All methods which employed Unconsolidated Undrained or Unconfined Compression Test data resulted in an overprediction of the initial settlement due to the extremely low Young's modulus obtained from those tests. With the exception of the Alibey cofferdam, all predictions which utilized moduli obtained from Consolidate Undrained Triaxial Tests, resulted in more accurate forecasts of initial settlement than those predictions using Unconsolidated moduli. This is shown in Figure 3-36. Nevertheless, within each case study, there existed some variation among the different prediction methods used. Figure 3-37 portrays this fact. Because of this variation, no generalized trends could be established regarding the methods themselves.

The comparison between predictions of total settlement and time rate of settlement and the observed performance is very difficult with the data available. Without analyzing pore pressure data to predict the average degree of consolidation at a certain time, comparison of the predictions and measurements is almost impossible. Only in cases where it is obvious from settlement versus log time plots that primary consolidation has essentially ended, can the comparison be valid.

The difference between the One-Dimensional and the Two-Dimensional Methods of predicting the time rate of

settlement is also difficult to evaluate. Because of the uncertainty of the input parameters and the large variations in total settlement predictions, the preference of one method over the other cannot be justified.

CHAPTER IV

CONCLUSIONS AND RECOMMENDATIONS

As stated in Chapter I, the goals of this thesis are to evaluate the use of standardized methods of predicting settlement in a soil-rock system and to evaluate the role of engineering judgement in geotechnical predictions. In order to accomplish this, it was necessary to develop standardized methods and a uniform procedure for selecting the related parameters. These methods and parameters were then combined to predict the performance of seven case studies. The predictions were then compared with actual measured performance and the comparisons were analyzed.

Many difficulties were encountered in making and analyzing the comparisons between observed and predicted settlement. Some of them are listed below.

1. In some instances, parameters had to be extrapolated from relatively distant borings. In others, the amount of data available for determing parameters was limited. The result was that some of the parameters could not be obtained according to the most preferred procedure.

2. Some projects have recorded their performance data for only a short period of time. Comparing long term measured and predicted performance in these cases was difficult.
3. A simple but valid procedure for obtaining a representative value of c_v is not well defined.

By attempting to overcome these difficulties in order to compare predicted and observed performance, the author has drawn the following conclusions:

1. The use of standardized methods to predict settlement is indeed feasible. Nevertheless, standardized procedures for selecting the related parameters are not.
2. Engineering judgement is required to make consistently accurate settlement predictions. The judgement should be exercised when selecting the appropriate parameters for use with a specific method.
3. A valid comparison between observed and predicted long-term settlement is not possible unless primary consolidation has ended or the average degree of

consolidation can be determined. A comparison of initial settlements is feasible, however.

4. In analyzing the comparison it is impossible to evaluate the method or the parameters separately. The method and parameters act as a unit and can only be evaluated together.
5. The value of the coefficient of consolidation obtained from oedometer tests does not yield accurate results when used with the methods specified.

Based on these results, the following recommendations for further study are made:

1. The major emphasis in making settlement predictions should be placed on the determination of the parameters. Relatively simple methods of analysis will yield accurate predictions when combined with the appropriate parameters. Engineering judgement should be used to aid in the selection of these parameters.
2. The average degree of consolidation should be determined so that time rate and long term settlement predictions may be

properly evaluated.

3. The possibility of using an empirical method to determine the coefficient of consolidation should be investigated. Its use with the specified methods would be at least as good as the use of an oedometer c_v.
4. A statisitcal study of standardized methods and the various procedures for obtaining parameters may yield more definitive results. With an estimate of the uncertainty involved, an engineer may be able to determine with reasonable confidence the range in which his predictions may lie. He may also be able to determine which procedure of selecting parameters best suits a particular method of analysis.

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TABLE 3-1

FUNDAMENTAL PARAMETERS

Case Study	1-D & Mod 1-D		Elastic		Local Yield			S-B	Time Rate	
	CR	RR	E_i	E_c	q_s/q_{ult}	F	S_R	μ(ave)	c_v	H_d
Alibey Test Embankment	0.506	0.067	1910 TSM	637 TSM	0.69	0.53	0.70	0.85	4.0×10^{-4} cm^2/sec	9.4 M
I-95 Sta 233+50	0.258	0.025	42.5	14.2 TSF	0.50	0.61	0.70	0.70	6.1×10^{-2} ft^2/day	63.8 ft.
I-95 Sta 362+50	0.244	0.023	22.8 TSF	7.6 TSF	0.83	0.32	0.85	0.60	5.8×10^{-2} ft^2/day	33.3 ft.
I-95 Sta 276+00	0.195	0.016	65.0	21.7 TSF	0.36	0.13	1.00	0.50	9.6×10^{-2} ft^2/day	15.3 ft.
MIT Test Section	0.173	0.016	187 TSF	62.3 TSF	0.48	0.35	0.92	0.65	14.9×10^{-2} ft^2/day	55.2 ft.
North-east Test Embank.	0.186	0.021	525 TSF	89 TSF	0.56	0.20	1.00	0.71	30.5×10^{-2} ft^2/day	41.6 ft.
Lagunillas Preload	.333	0.042	47.0 TSF	18.3 TSF	$q_s/q_{ult}>1$ Failure			0.95	2.2×10^{-2} ft^2/day	7.7 ft.

TABLE 3-2
TOTAL SETTLEMENT PREDICTIONS

Case Study	1 - D		MOD 1 - D		ELASTIC		Elastic w/ Loc. Yield	Skem. - Bjerrum
	ρ_{SAND}	ρ_{CF}	ρ_{SAND}	ρ_{CF}	ρ_{e}	ρ_{CF}	ρ_{i}	ρ_{CF}
Alibey Test Embankment	-	2.54 M	-	1.96M	0.03M	0.033M	0.05M	1.67M
I-95 Sta 233+50	-	8.53 ft.	-	7.22 ft.	2.89 ft	16.10 ft	4.10 ft	5.06 ft.
I-95 Sta 263+00	0.18 ft	5.98 ft.	0.18 ft.	5.83 ft.	4.21 ft.	22.60 ft.	4.95 ft.	3.50 ft.
I-95 Sta 276+00	0.31 ft.	1.24 ft.	0.31 ft.	1.17 ft.	0.28 ft.	3.53 ft.	0.28 ft.	0.59 ft.
M.I.T. Test Section	0.19 ft.	4.79 ft.	0.19 ft.	4.24 ft.	0.60 ft.	3.61.ft.	0.72 ft.	2.76 ft.
Northeast Test Embank.	0.19 ft	3.59 ft.	0.19 ft.	3.54 ft.	0.30 ft.	1.70 ft.	0.30 ft.	2.50 ft.
Lagunillas Preload	0.39 ft.	3.57 ft.	0.39 ft.	3.49 ft.	0.56 ft.	2.34 ft.	Failure	3.33 ft.

TABLE 3-3
TIME RATE OF SETTLEMENT PREDICTIONS

One Dimensional Consolidation Method

U%	Predicted Time (days)						
	Alibey Dam	I-95 233+50	I-95 263+00	I-95 276+00	MIT Test Sec.	NE Test Section	Lagun. Preload
10	206	535	153	19	163	45	22
30	1820	4740	1357	166	1450	403	192
50	-	-	-	-	-	-	-
70	10,400	27,000	7710	945	8240	2290	1093
90	21,800	56,700	16,200	1986	17,300	4820	2300
95	28,900	75,200	21,500	2,630	23,000	6390	3050

Two-Dimensional Consolidation Method

U%	Predicted Time (days)						
	Alibey Dam	I-95 233+50	I-95 263+00	I-95 276+00	MIT Test Section	NE Test Section	Lag. Preload
10	412	1070	306	37	326	91	43
50	6170	14,000	4580	562	4480	1362	650
75	13,600	31,400	10,120	1240	10,000	3005	1436
90	23,100	54,700	17,180	2104	17,300	5110	2440

TABLE 3-4

COMPARISON BETWEEN INITIAL SETTLEMENTS OBSERVED AND PREDICTED

Case	Measurement	Observed Settlement	Skempton-Bjerrum	Skempton-Bjerrum w/ Local Yield	Elastic	Elastic w/ Local Yield	Major Source of Undrained Modulules
Alibey	Meters	0.53	6.06	0.08	0.05	0.07	$\overline{CK_oUC}$
I-95 Sta 233+50	Feet	2.00	3.16	4.35	3.72	4.88	UU
I-95 Sta 263+00	Feet	1.63	4.60	5.12	2.36	2.90	UU + U
I-95 Sta 276+00	Feet	0.25	0.38	0.38	0.90	0.90	UU + $\overline{CIU}$
MIT Test Section	Feet	1.38	1.16	1.22	1.28	1.34	$\overline{CIUC}$ $\overline{CK_oUC}$
North-east Test Embank. PL1	Feet	0.58	0.51	0.51	0.44	0.44	$\overline{CK_oUC}$
PL2		0.58					
PL3		0.67					
Lagunilla Preload	Feet	1.83	1.03	1.03	0.90	0.90	$\overline{CIUC}$

Alibey Test Embankment
Idealized Cross Section and Stress History

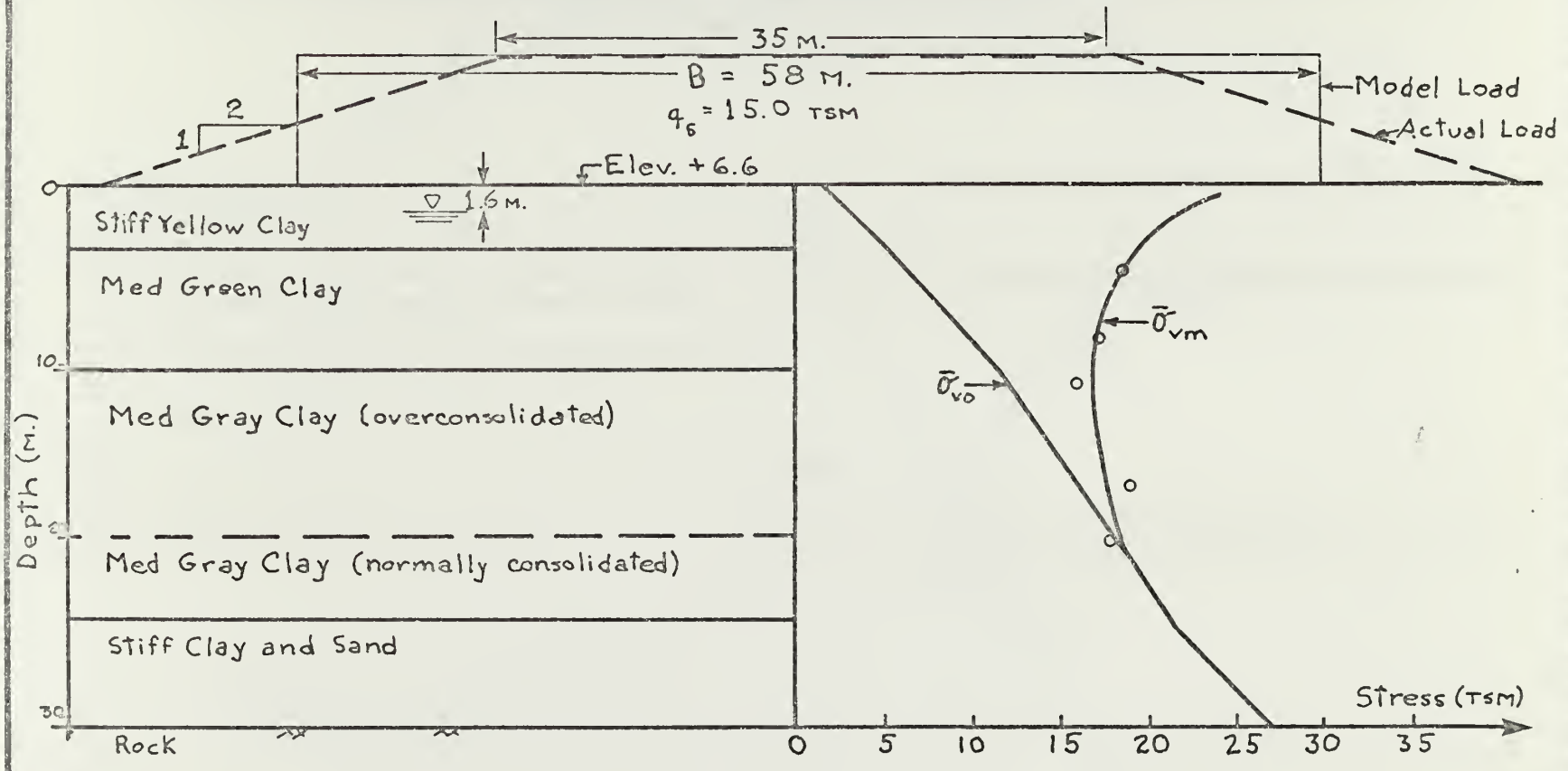


Figure 3-1.

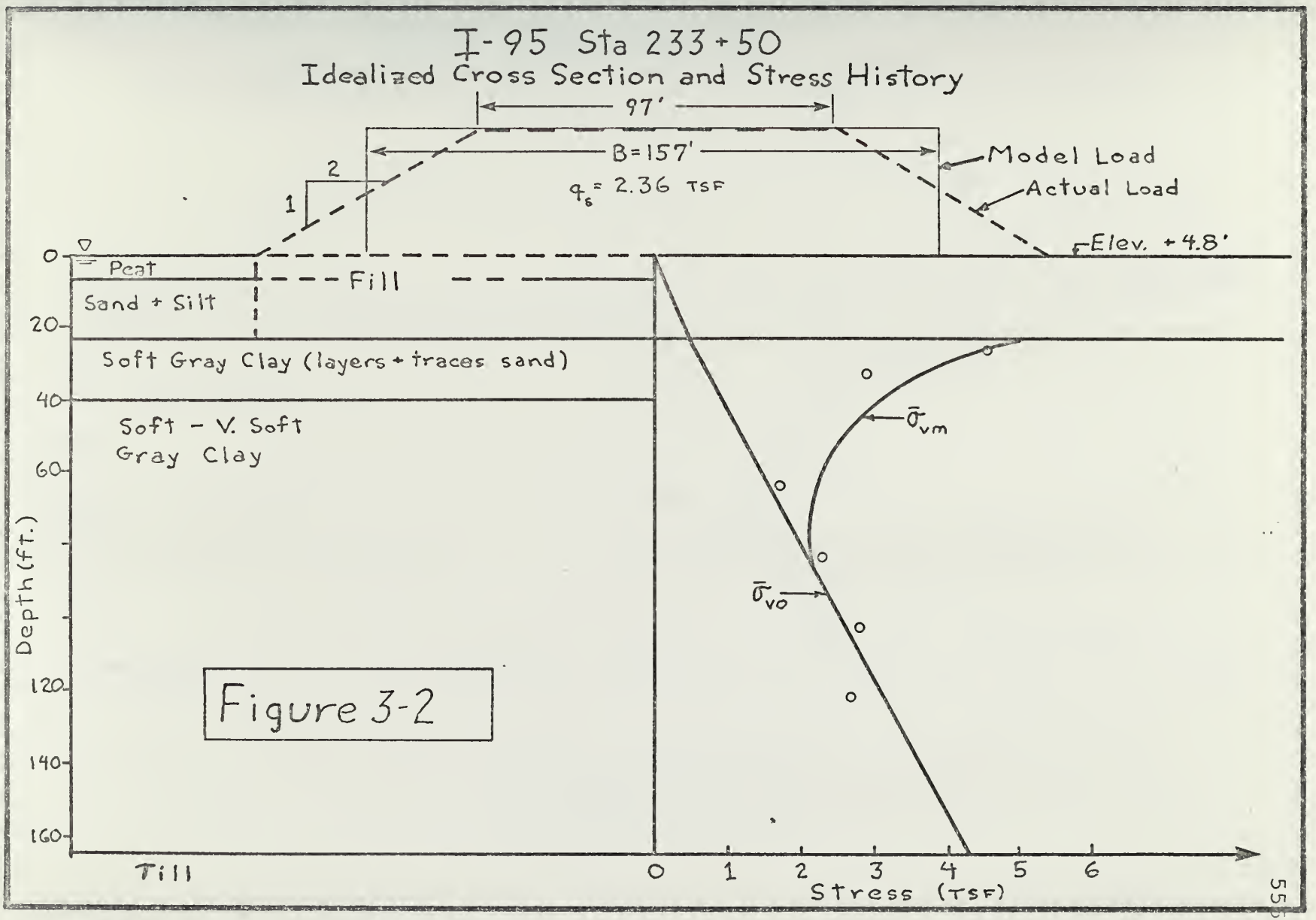

I-95 Sta 233+50
Idealized Cross Section and Stress History
97'
B=157'
Model Load
Actual Load
q_s = 2.36 TSF
2
1
Elev. +4.8'
Peat
Fill
Sand + Silt
Soft Gray Clay (layers + traces sand)
Soft - V. Soft
Gray Clay
Till
$\bar{\sigma}_{vm}$
$\bar{\sigma}_{vo}$
Depth (ft.)
0
20
40
60
120
140
160
0
1
2
3
4
5
6
Stress (TSF)

Figure 3-2

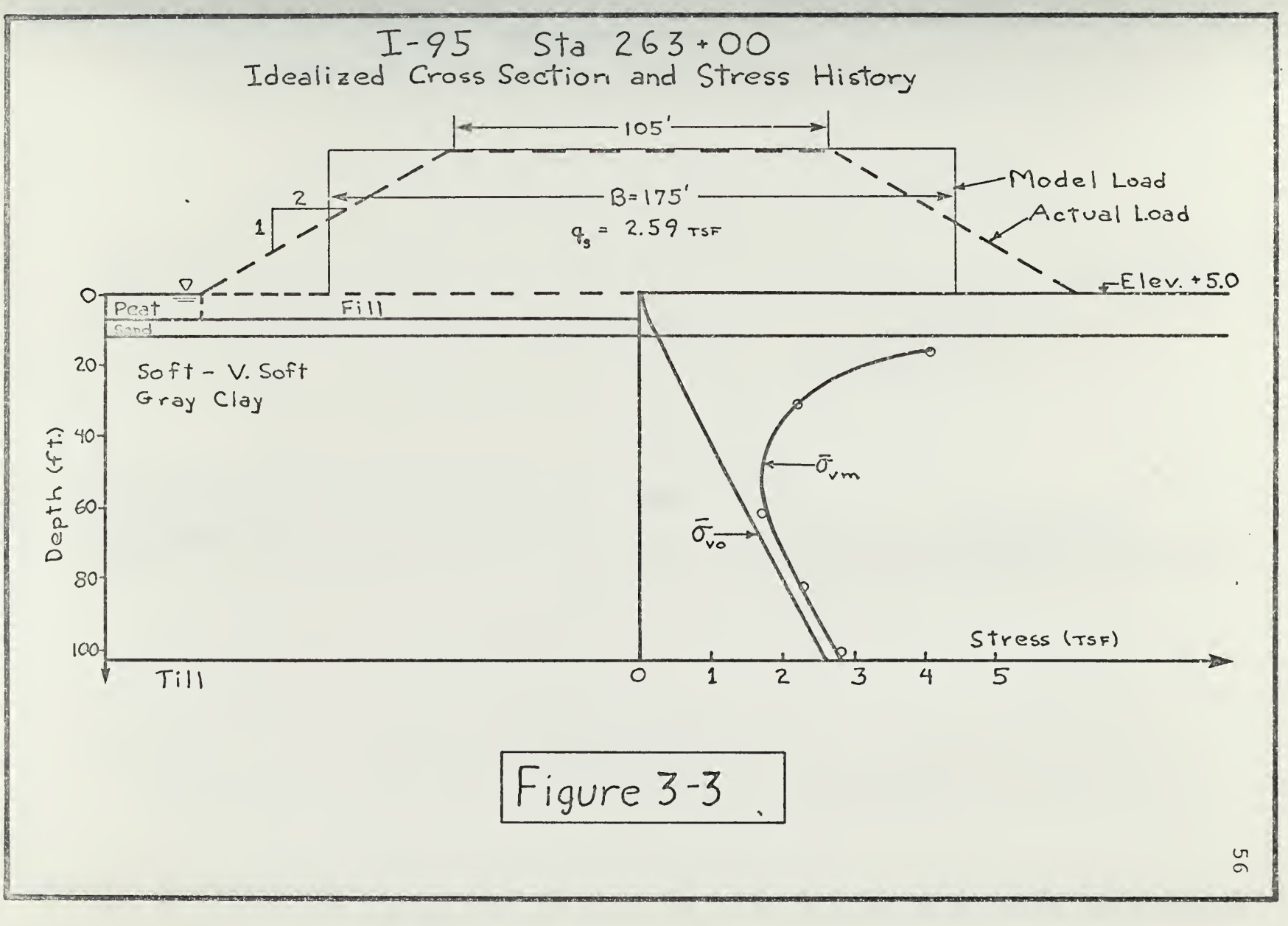

I-95 Sta 263+00
Idealized Cross Section and Stress History
105'
B=175'
q_s = 2.59 TSF
2
1
Model Load
Actual Load
Elev. +5.0
Peat
Sand
Fill
Soft - V. Soft
Gray Clay
Till
Depth (ft.)
0
20
40
60
80
100
$\bar{\sigma}_{vm}$
$\bar{\sigma}_{vo}$
Stress (TSF)
0
1
2
3
4
5

Figure 3-3

I-95 Sta 276+00
Idealized Cross Section and Stress History

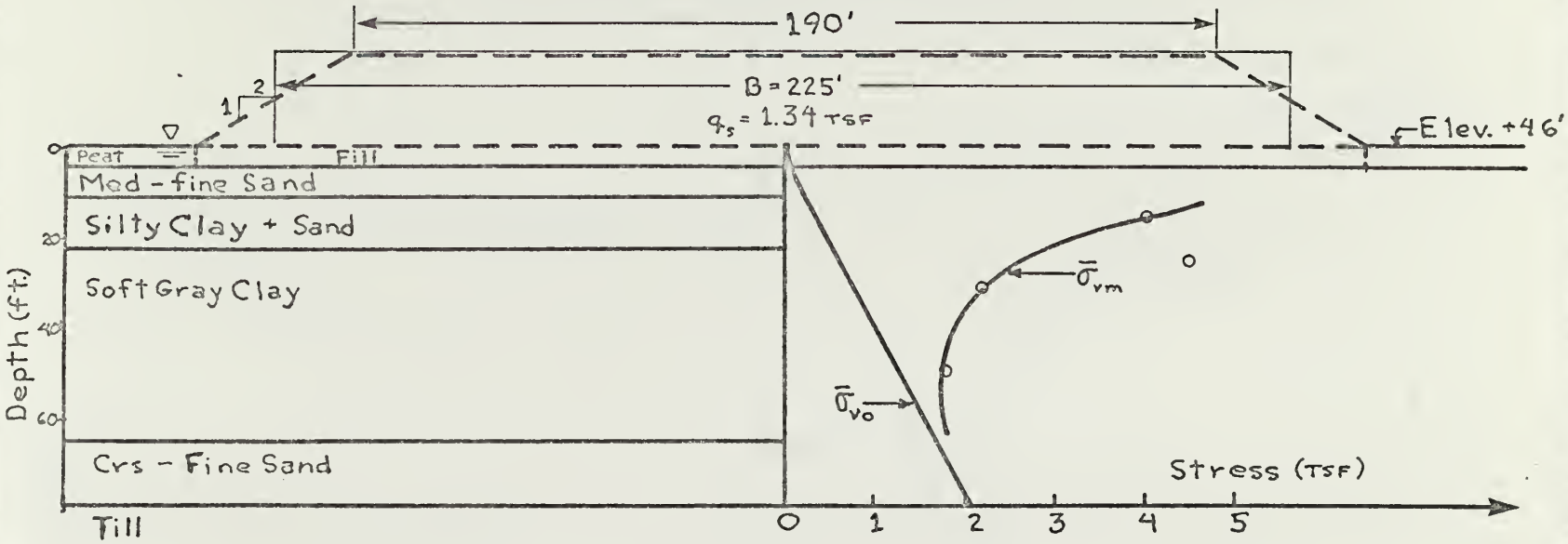


Figure 3-4

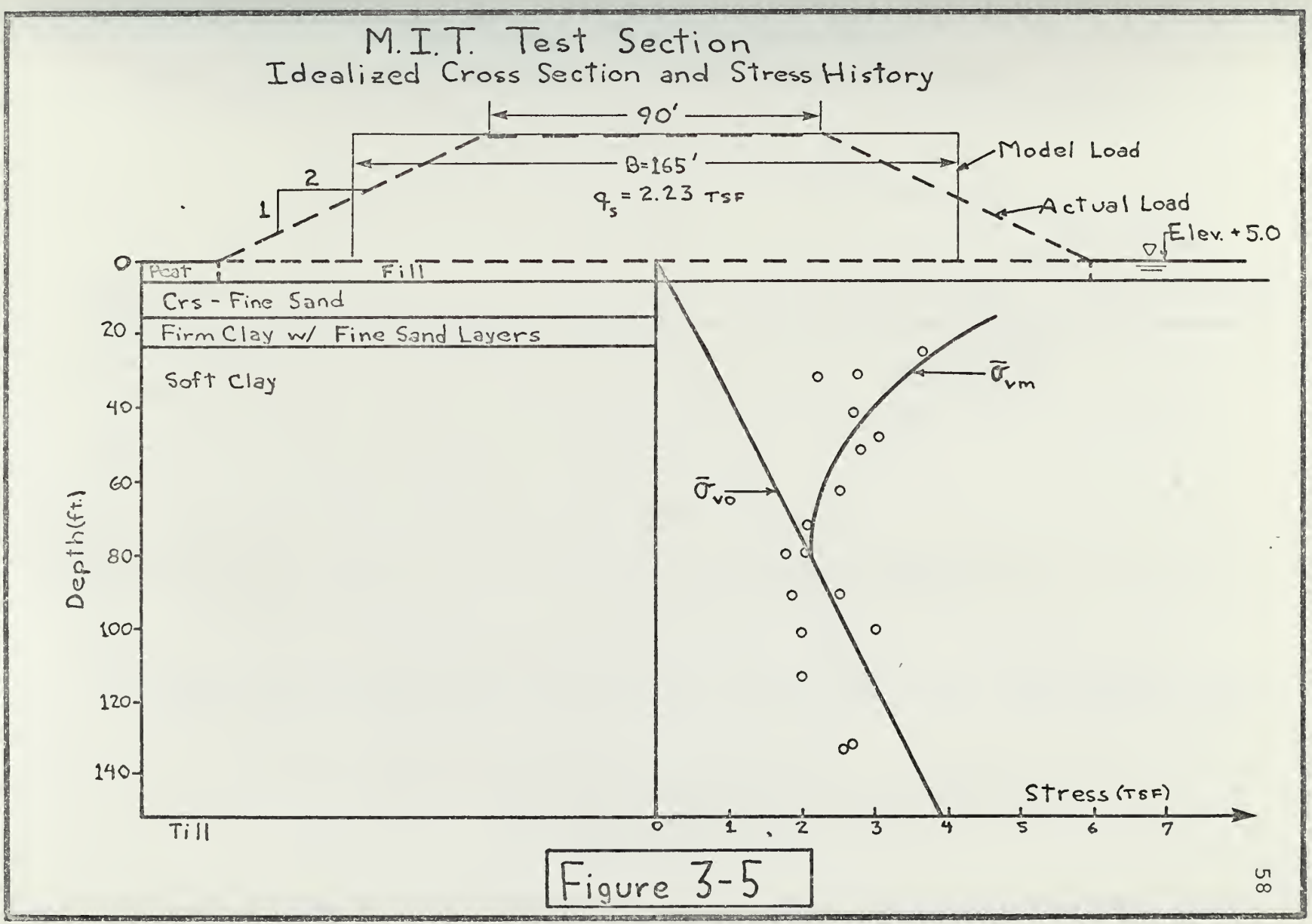

M.I.T. Test Section
Idealized Cross Section and Stress History
90'
B=165'
q_s = 2.23 TSF
Model Load
Actual Load
Elev. +5.0
2
1
Peat
Fill
Crs - Fine Sand
Firm Clay w/ Fine Sand Layers
Soft Clay
Till
Depth(ft.)
0
20
40
60
80
100
120
140
$\bar{\sigma}_{vm}$
$\bar{\sigma}_{vo}$
Stress (TSF)
0
1
2
3
4
5
6
7

Figure 3-5

Northeast Test Embankment
Idealized Cross Section and Stress History

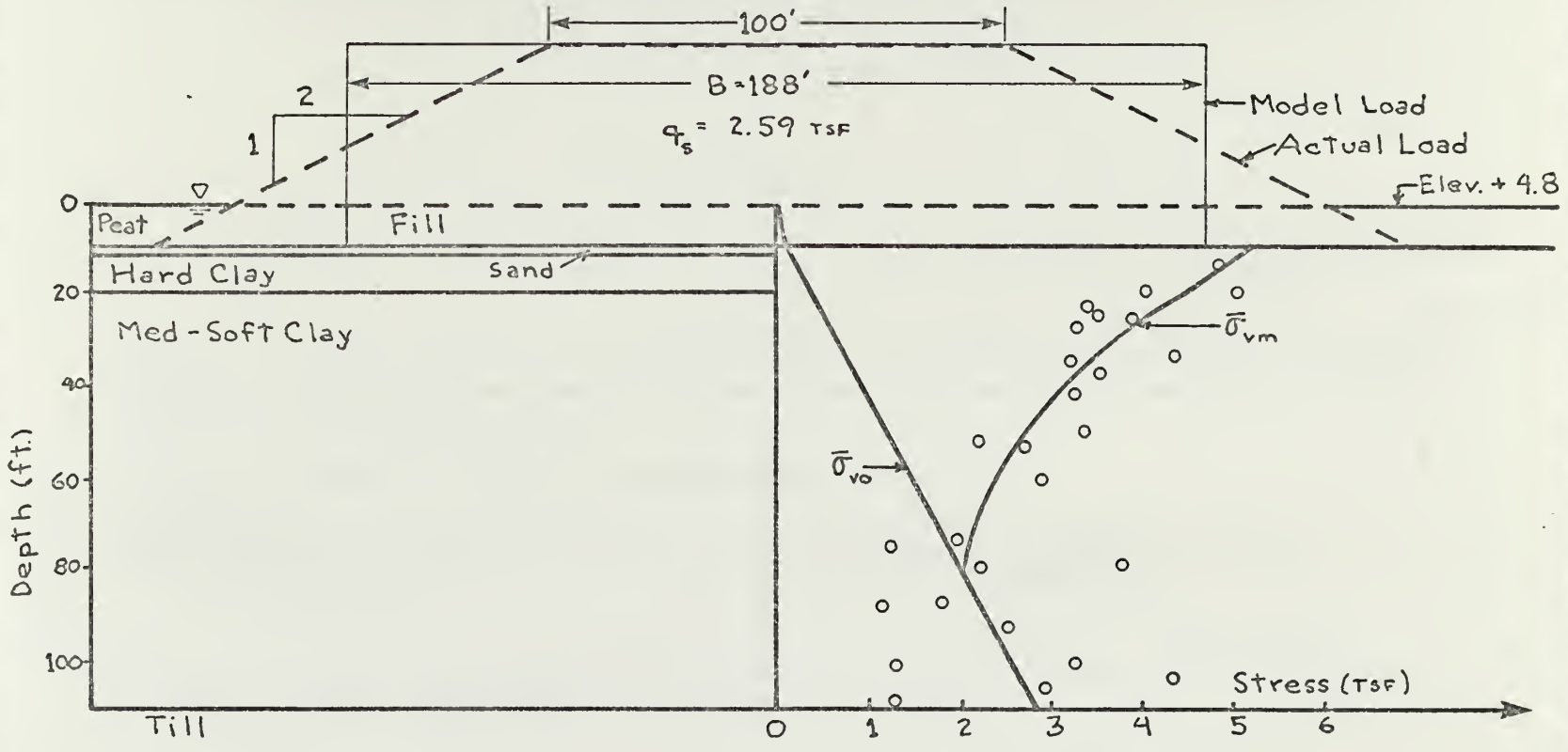


Figure 3-6.

Lagunillas Preload
Idealized Cross Section and Stress History

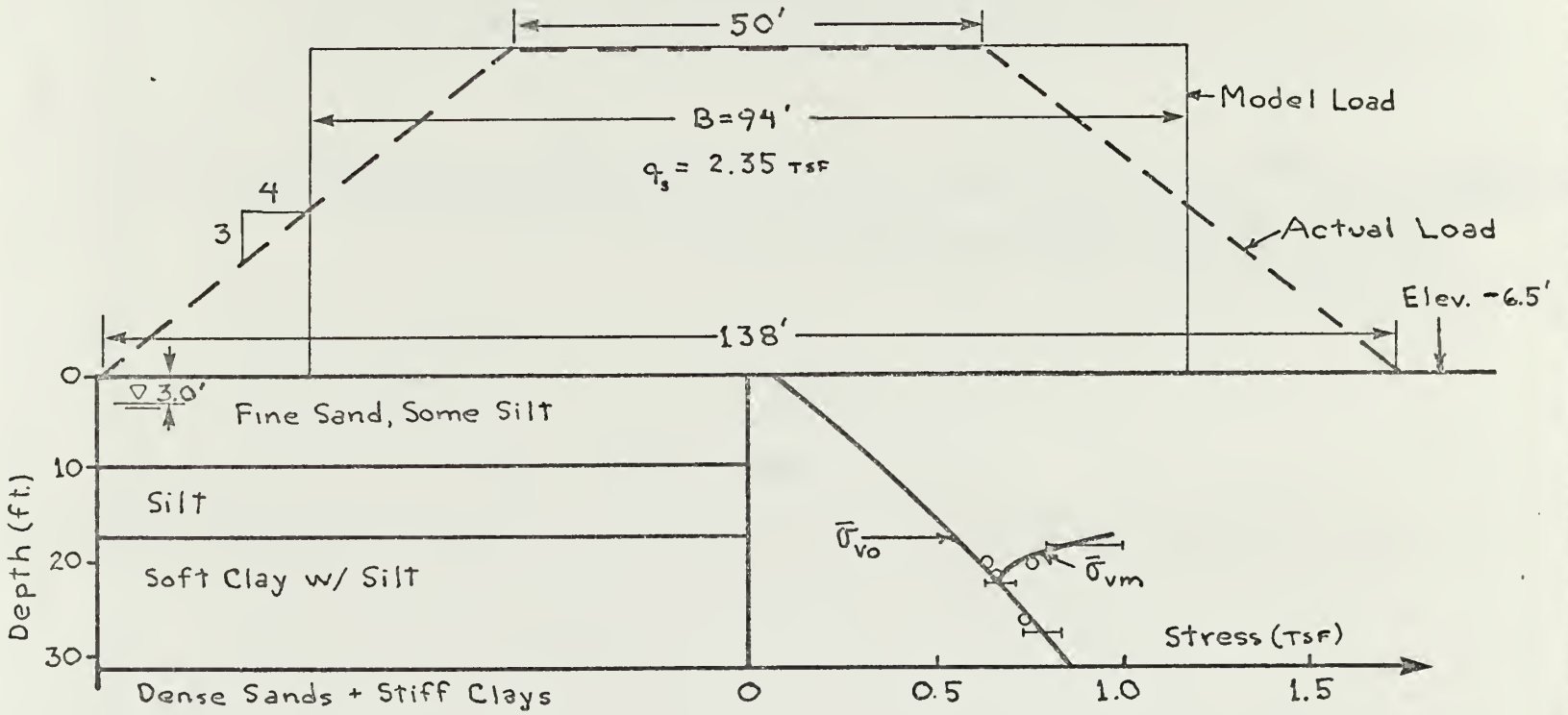


Figure 3-7

09

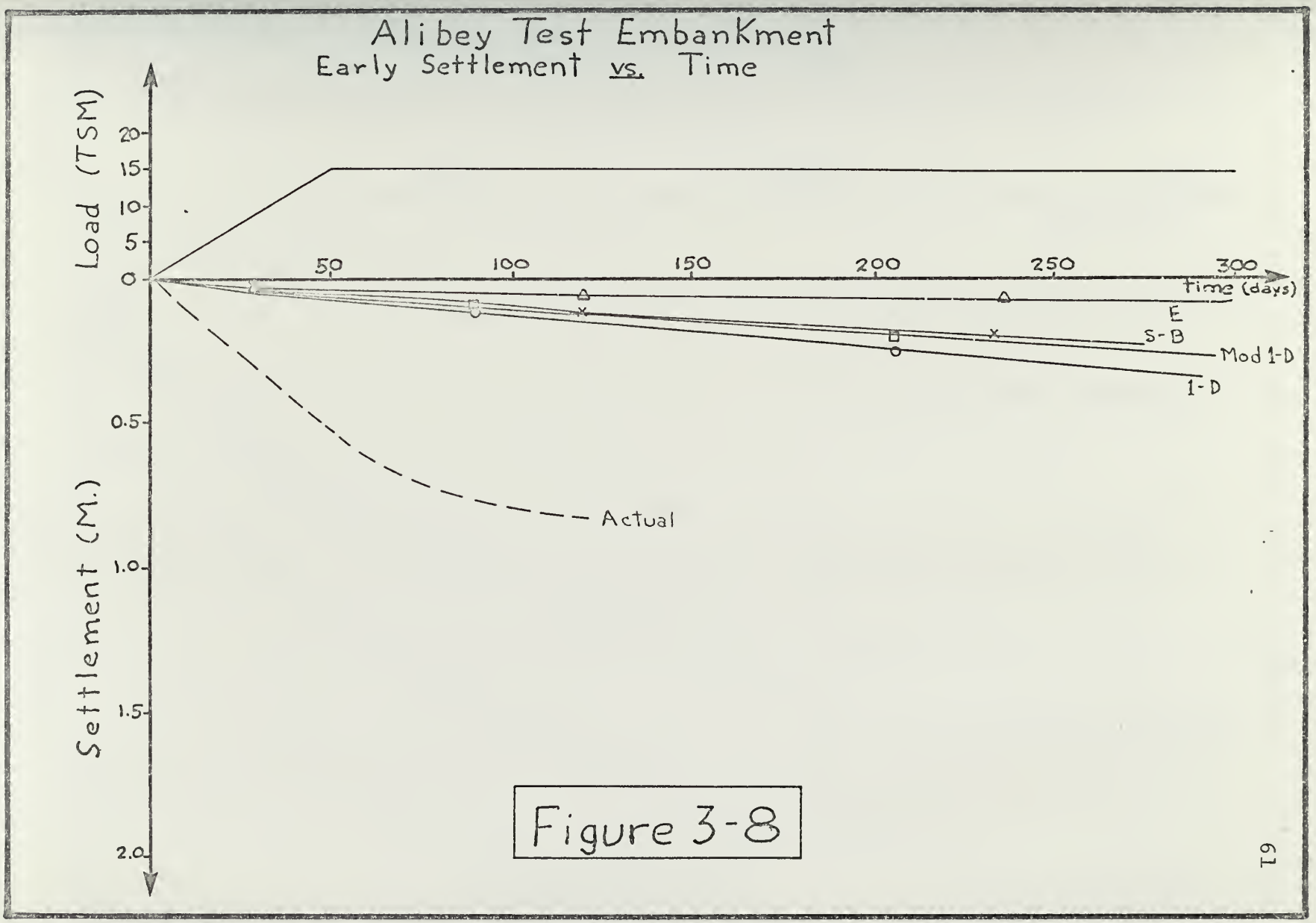

Alibey Test Embankment
Early Settlement vs. Time
Load (TSM)
20
15
10
5
0
50
100
150
200
250
300
Time (days)
E
S-B
Mod 1-D
1-D
0.5
Actual
Settlement (M.)
1.0
1.5
2.0

Figure 3-8

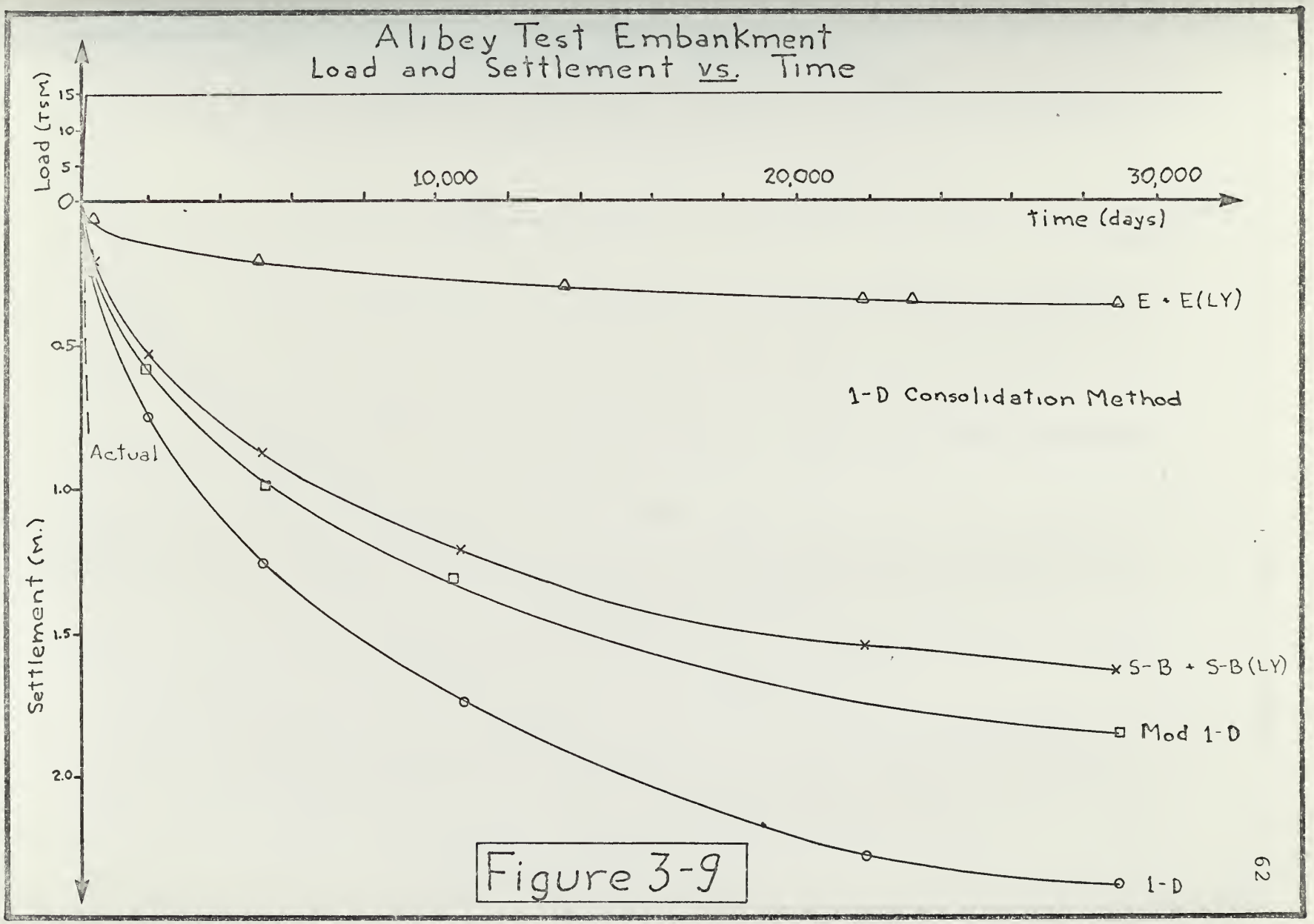

Alibey Test Embankment
Load and Settlement vs. Time
Load (TSM)
15
10
5
0
10,000
20,000
30,000
time (days)
E + E(LY)
0.5
1-D Consolidation Method
Actual
1.0
Settlement (m.)
1.5
S-B + S-B(LY)
Mod 1-D
2.0
1-D

Figure 3-9

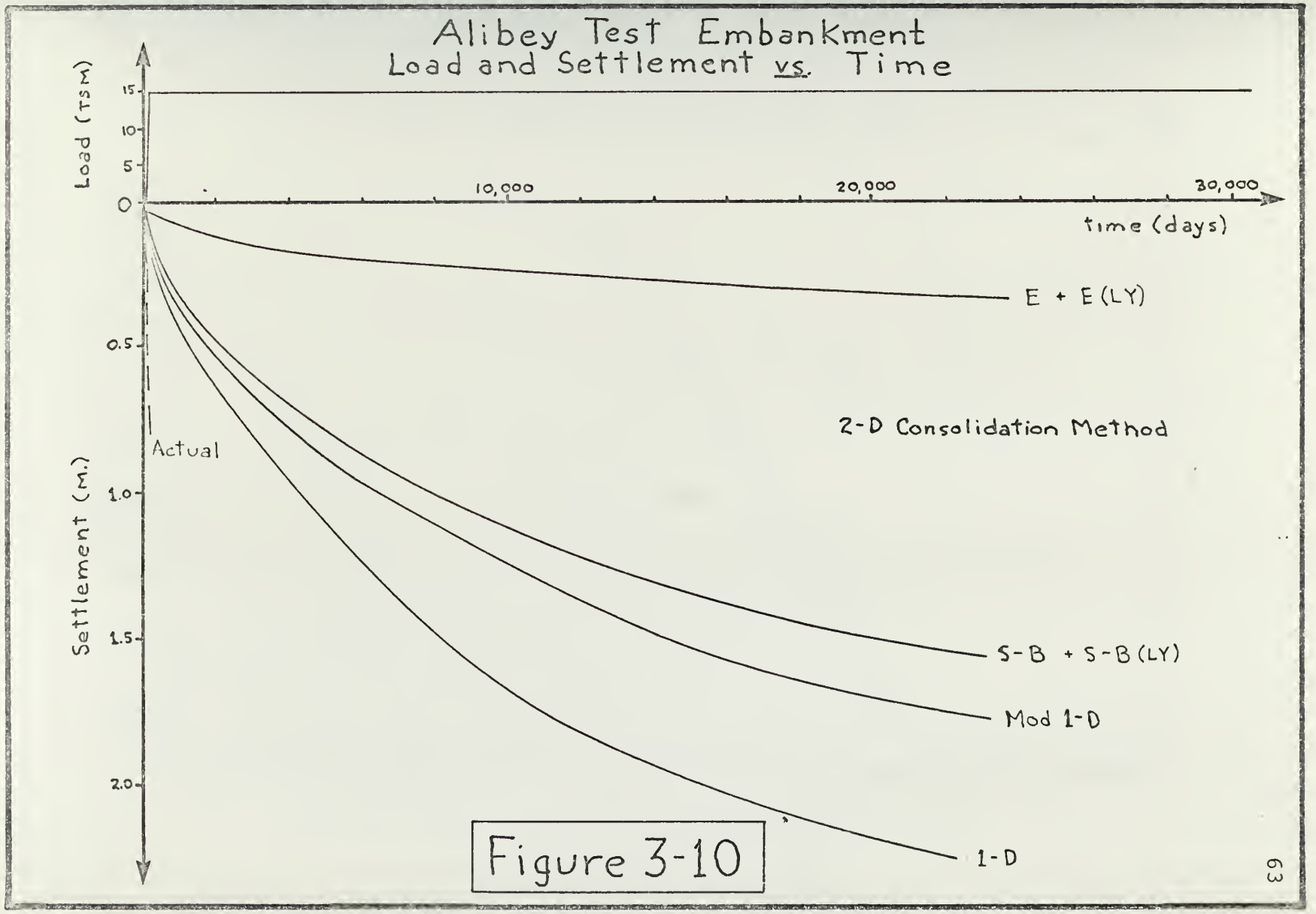

Alibey Test Embankment
Load and Settlement vs. Time
Load (TSM)
15
10
5
0
10,000
20,000
30,000
time (days)
E + E(LY)
0.5
Actual
2-D Consolidation Method
Settlement (M.)
1.0
1.5
S-B + S-B(LY)
Mod 1-D
2.0
1-D

Figure 3-10

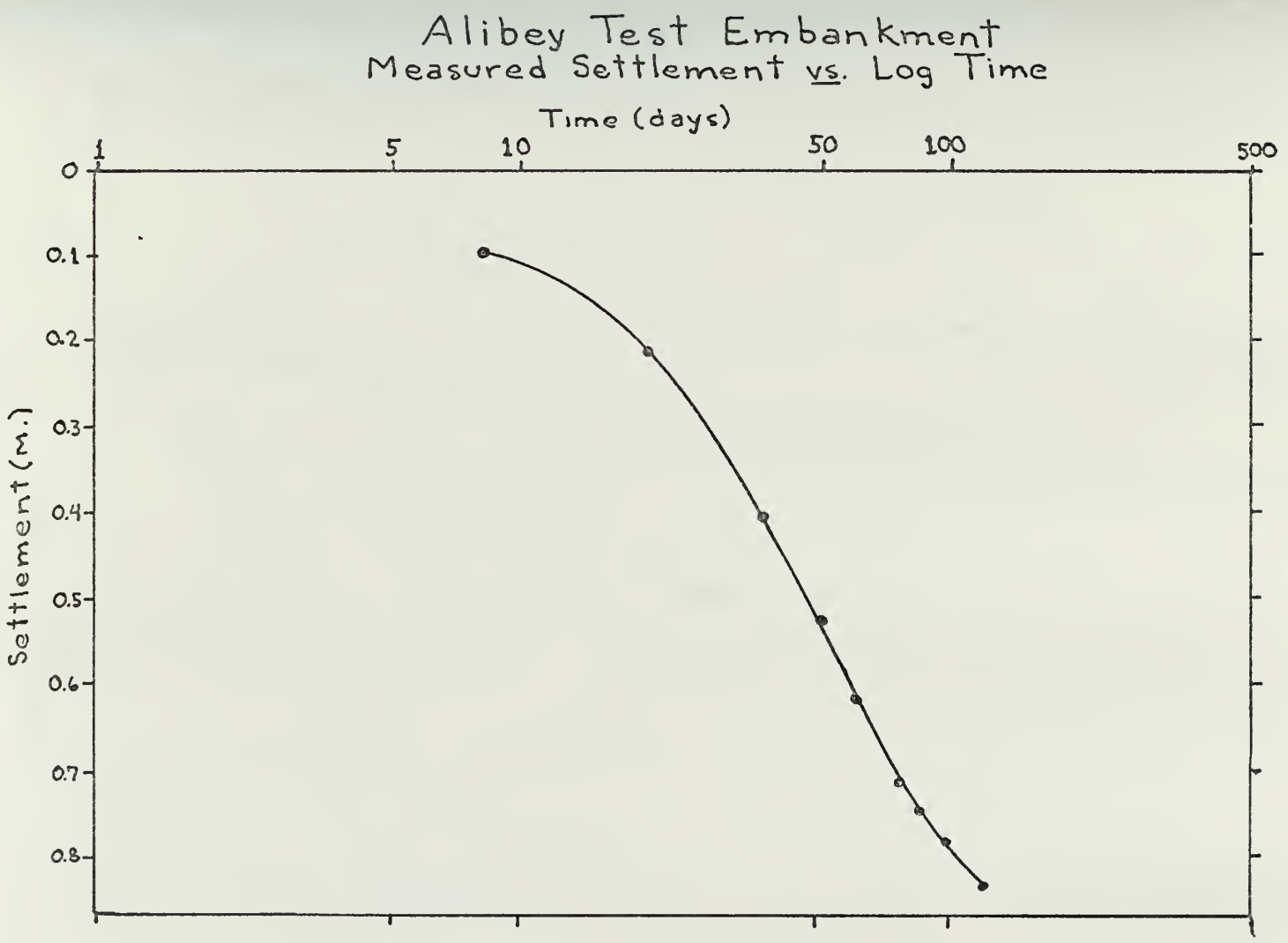

Alibey Test Embankment
Measured Settlement vs. Log Time
Time (days)
1
5
10
50
100
500
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Settlement (M.)

Figure 3-11

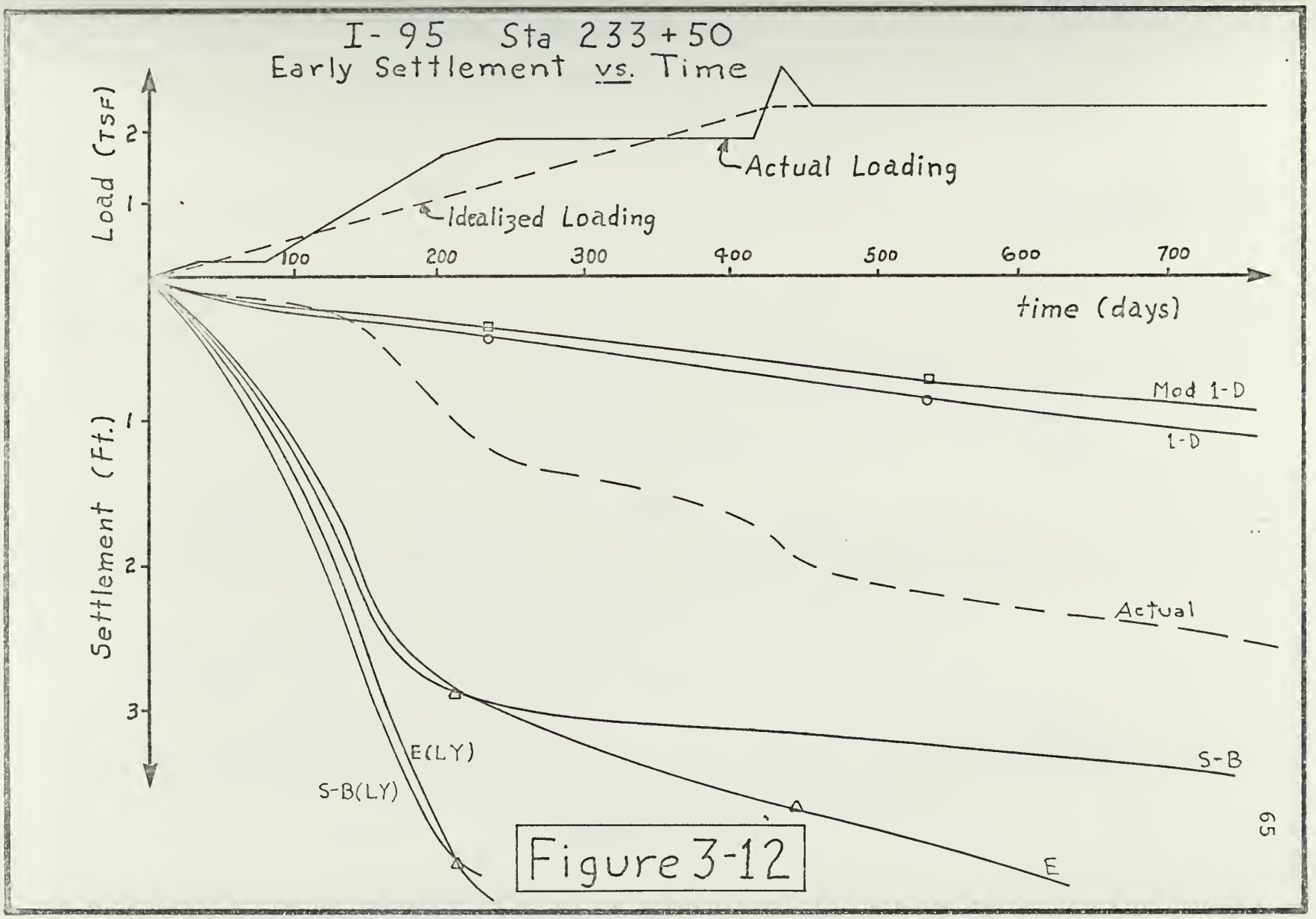

I-95 Sta 233+50
Early Settlement vs. Time
Load (TSF)
2
1
Actual Loading
Idealized Loading
100
200
300
400
500
600
700
time (days)
Mod 1-D
1-D
Settlement (Ft.)
1
2
3
Actual
S-B
E(LY)
S-B(LY)
E

Figure 3-12

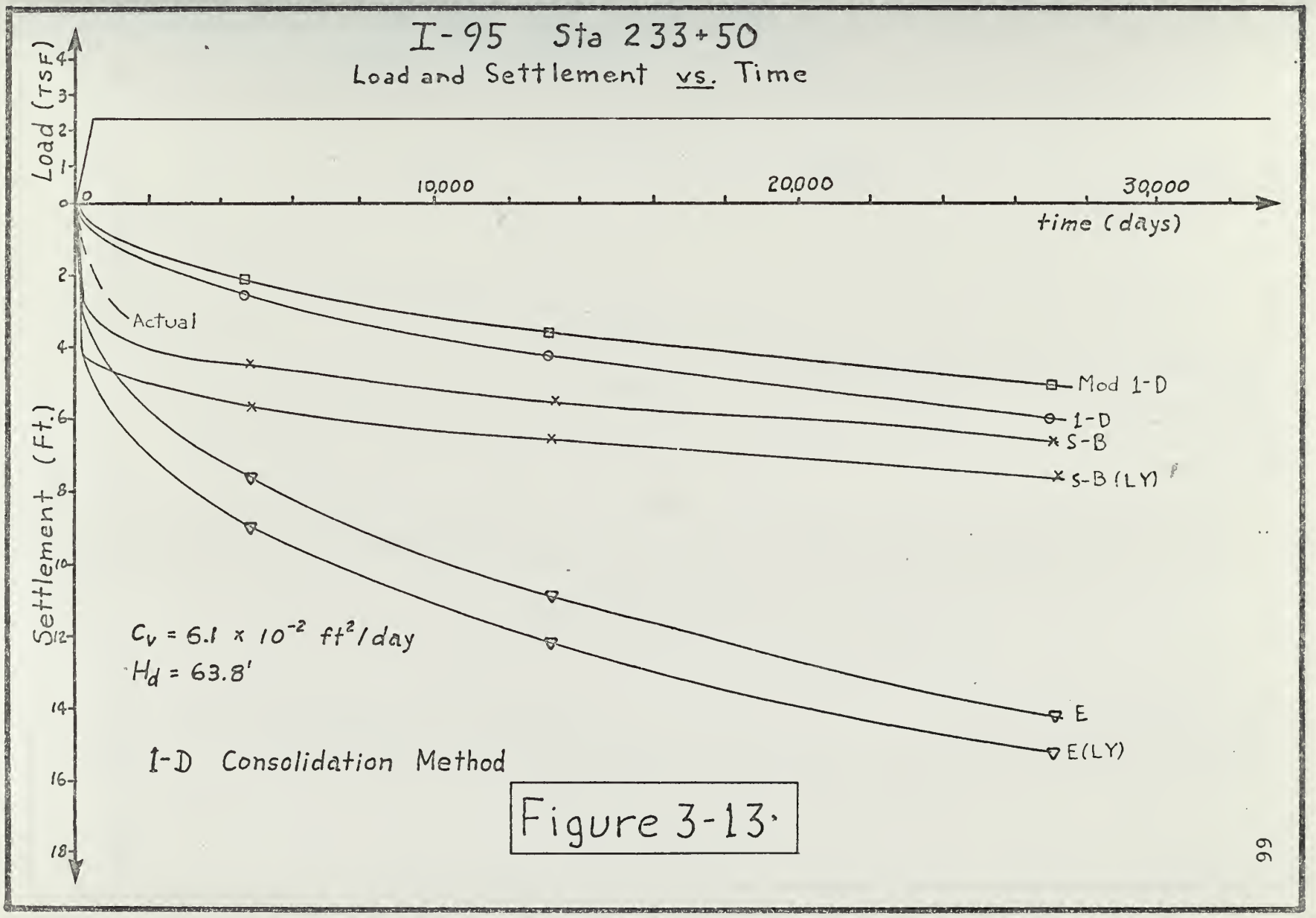

I-95 Sta 233+50
Load and Settlement vs. Time
Load (TSF)
Settlement (Ft.)
time (days)
10,000
20,000
30,000
Actual
Mod 1-D
1-D
S-B
S-B (LY)
E
E(LY)
$C_v = 6.1 \times 10^{-2}$ ft²/day
$H_d = 63.8'$
1-D Consolidation Method

Figure 3-13.

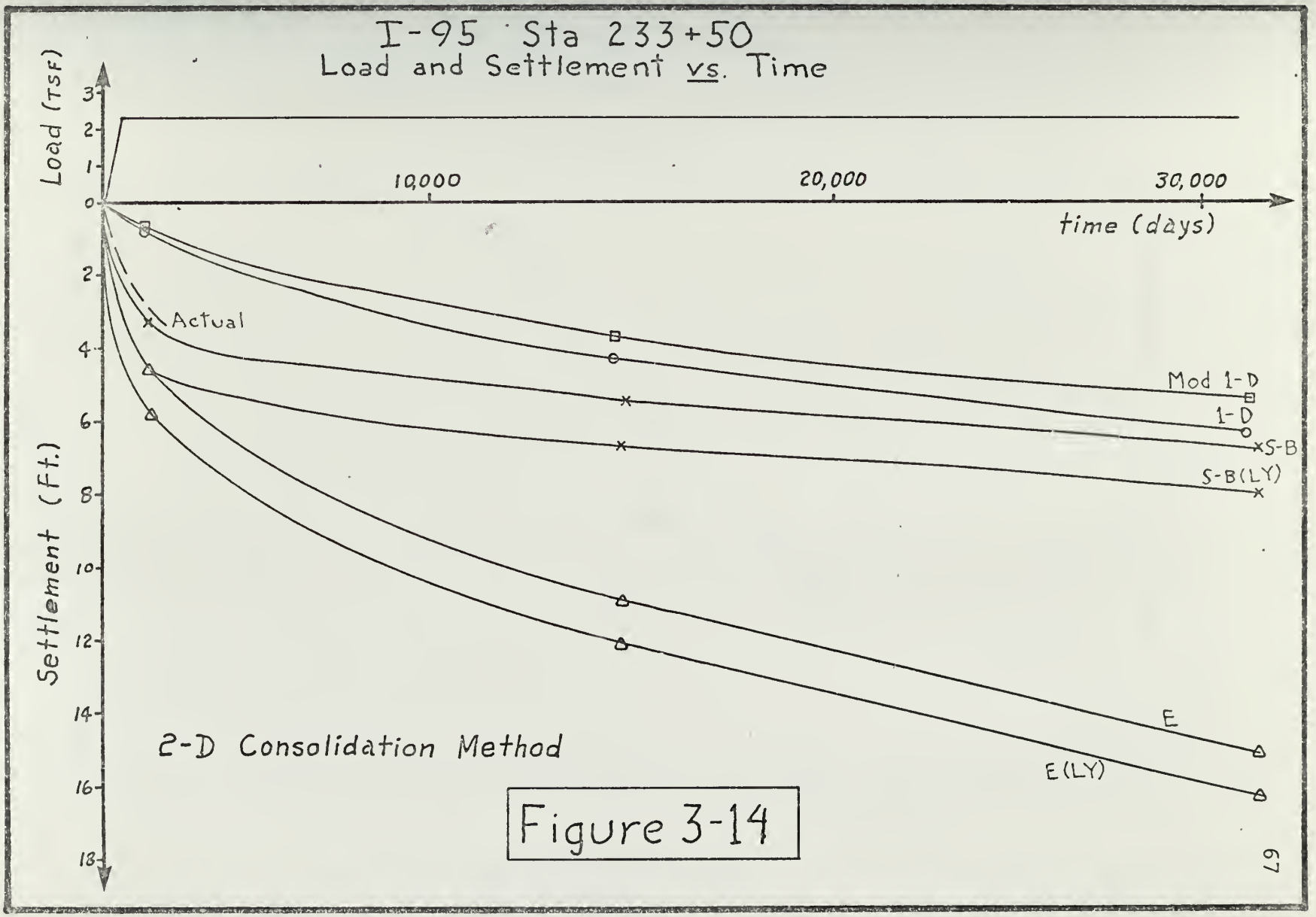

I-95 Sta 233+50
Load and Settlement vs. Time
Load (TSF)
3
2
1
0
10,000
20,000
30,000
time (days)
Actual
Mod 1-D
1-D
S-B
S-B(LY)
E
E(LY)
Settlement (Ft.)
2
4
6
8
10
12
14
16
18
2-D Consolidation Method

Figure 3-14

I-95 Sta 233+50

Measured Settlement vs. Log Time

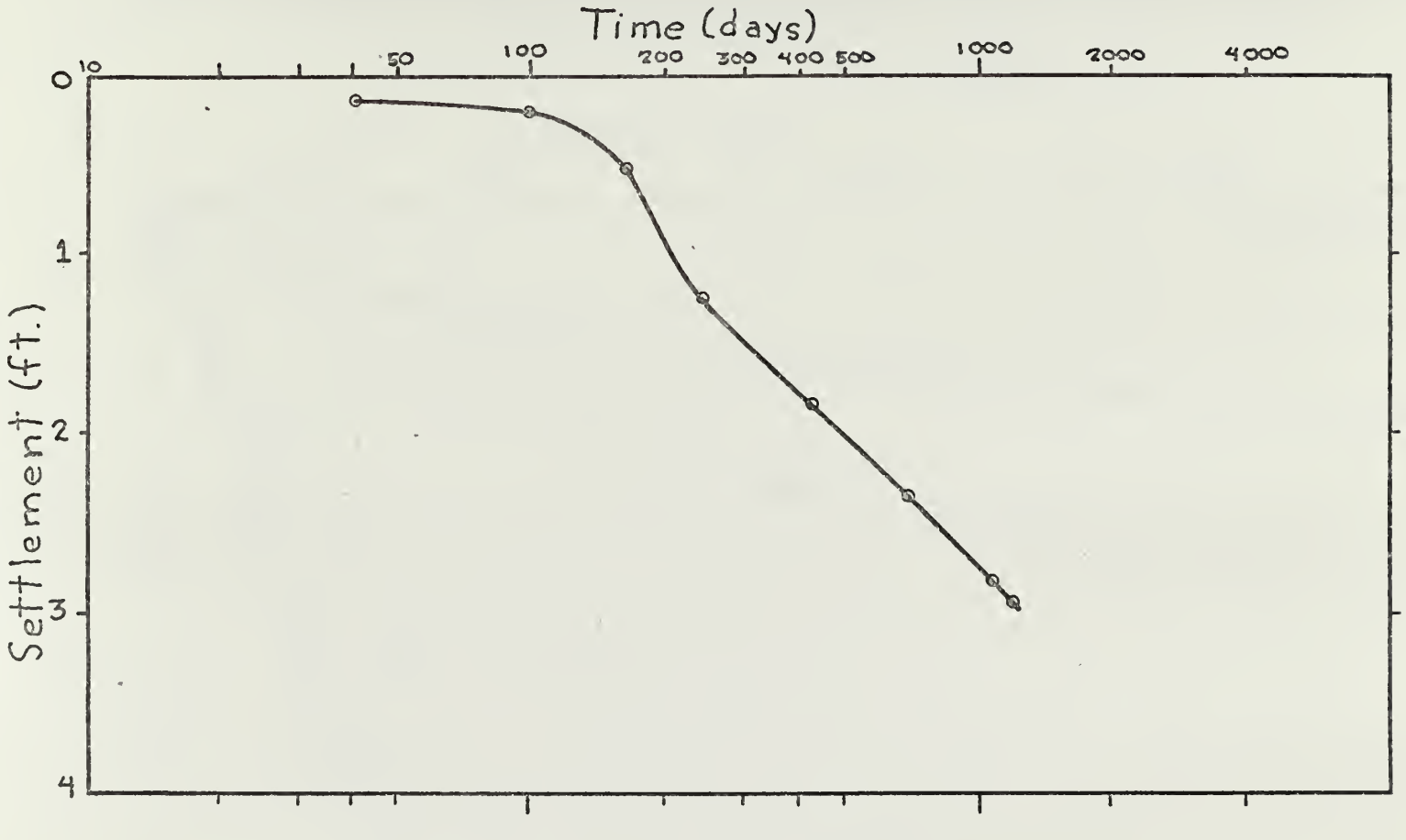


Figure 3-15

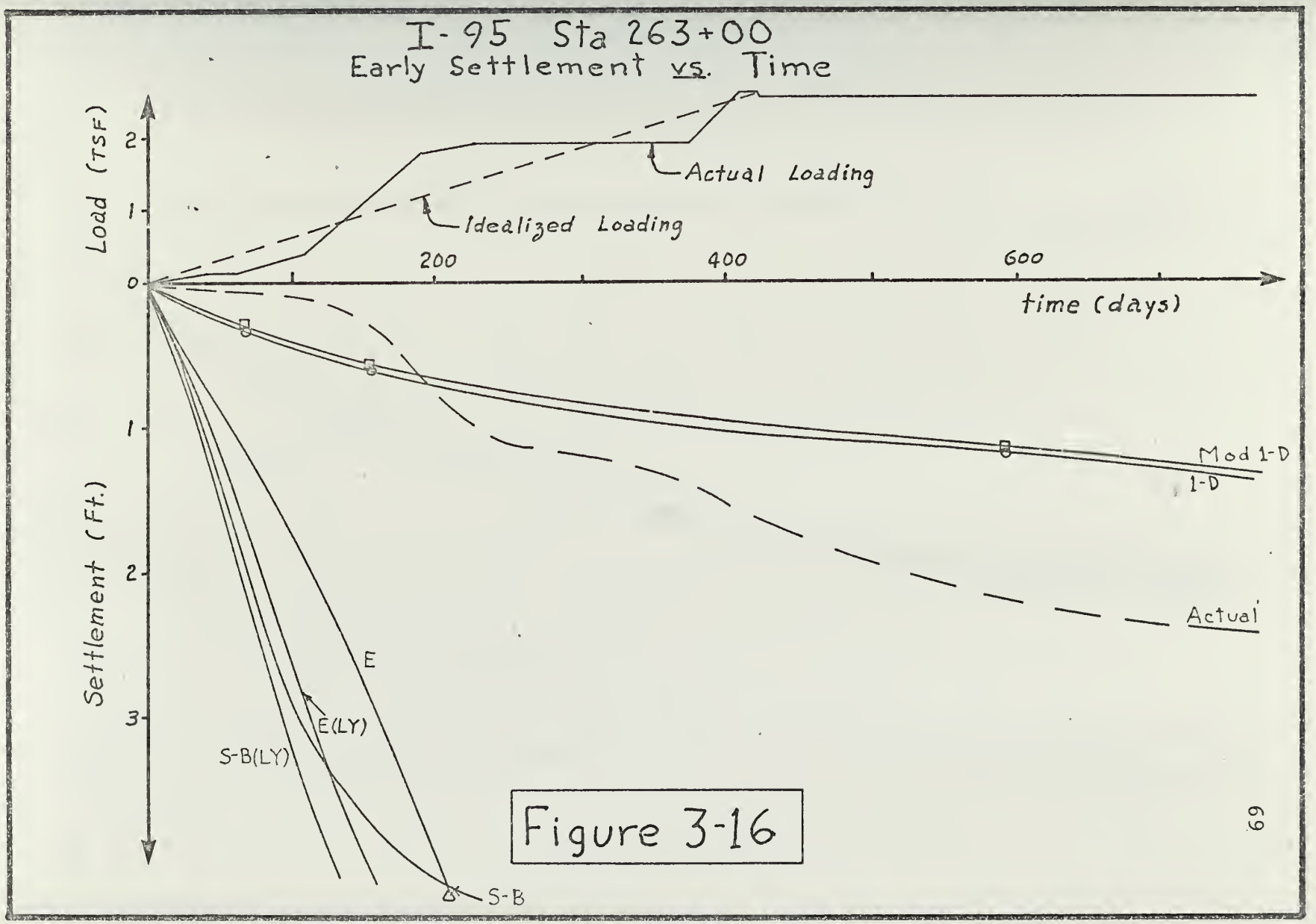

I-95 Sta 263+00
Early Settlement vs. Time
Load (TSF)
2
1
0
1
2
3
Settlement (Ft.)
200
400
600
time (days)
Actual Loading
Idealized Loading
Mod 1-D
1-D
Actual
E
E(LY)
S-B(LY)
S-B

Figure 3-16

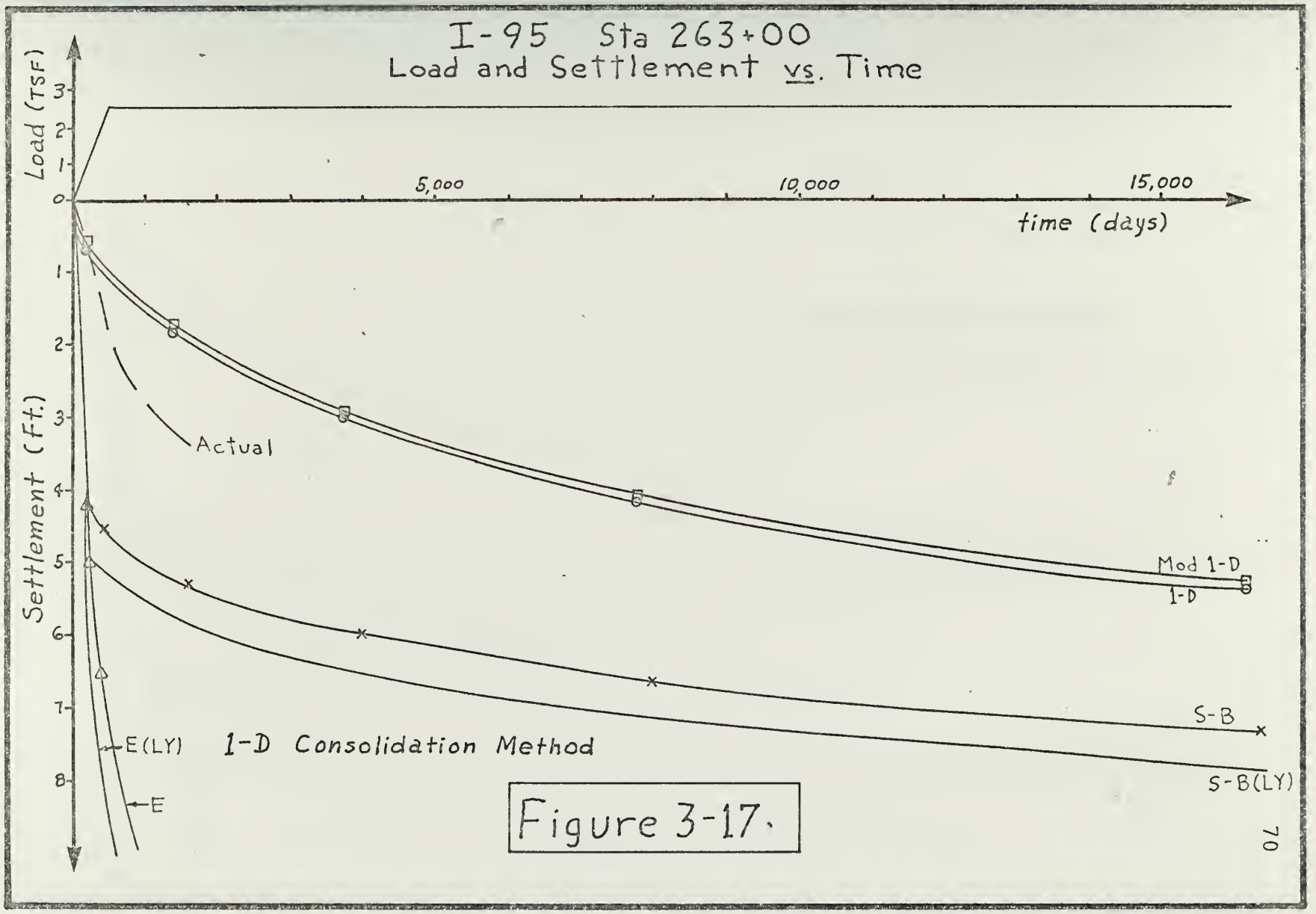

I-95 Sta 263+00
Load and Settlement vs. Time
Load (TSF)
0
1
2
3
5,000
10,000
15,000
time (days)
Settlement (Ft.)
1
2
3
4
5
6
7
8
Actual
Mod 1-D
1-D
S-B
S-B(LY)
E(LY)
E
1-D Consolidation Method

Figure 3-17.

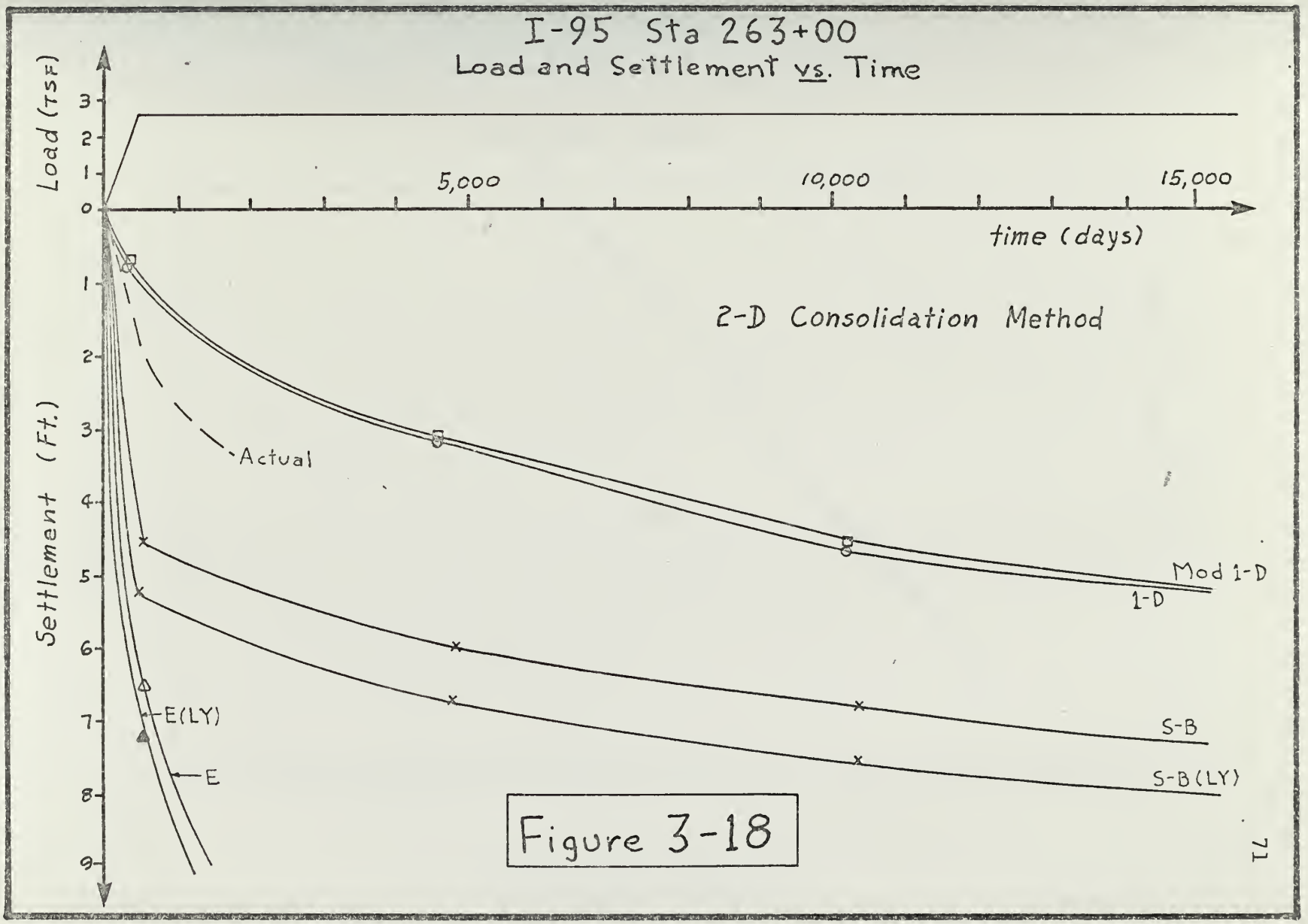

I-95 Sta 263+00
Load and Settlement vs. Time
Load (TSF)
0
1
2
3
5,000
10,000
15,000
time (days)
2-D Consolidation Method
Settlement (Ft.)
1
2
3
4
5
6
7
8
9
Actual
Mod 1-D
1-D
E(LY)
E
S-B
S-B(LY)

Figure 3-18

I-95 Sta 263+00

Measured Settlement vs. Log Time

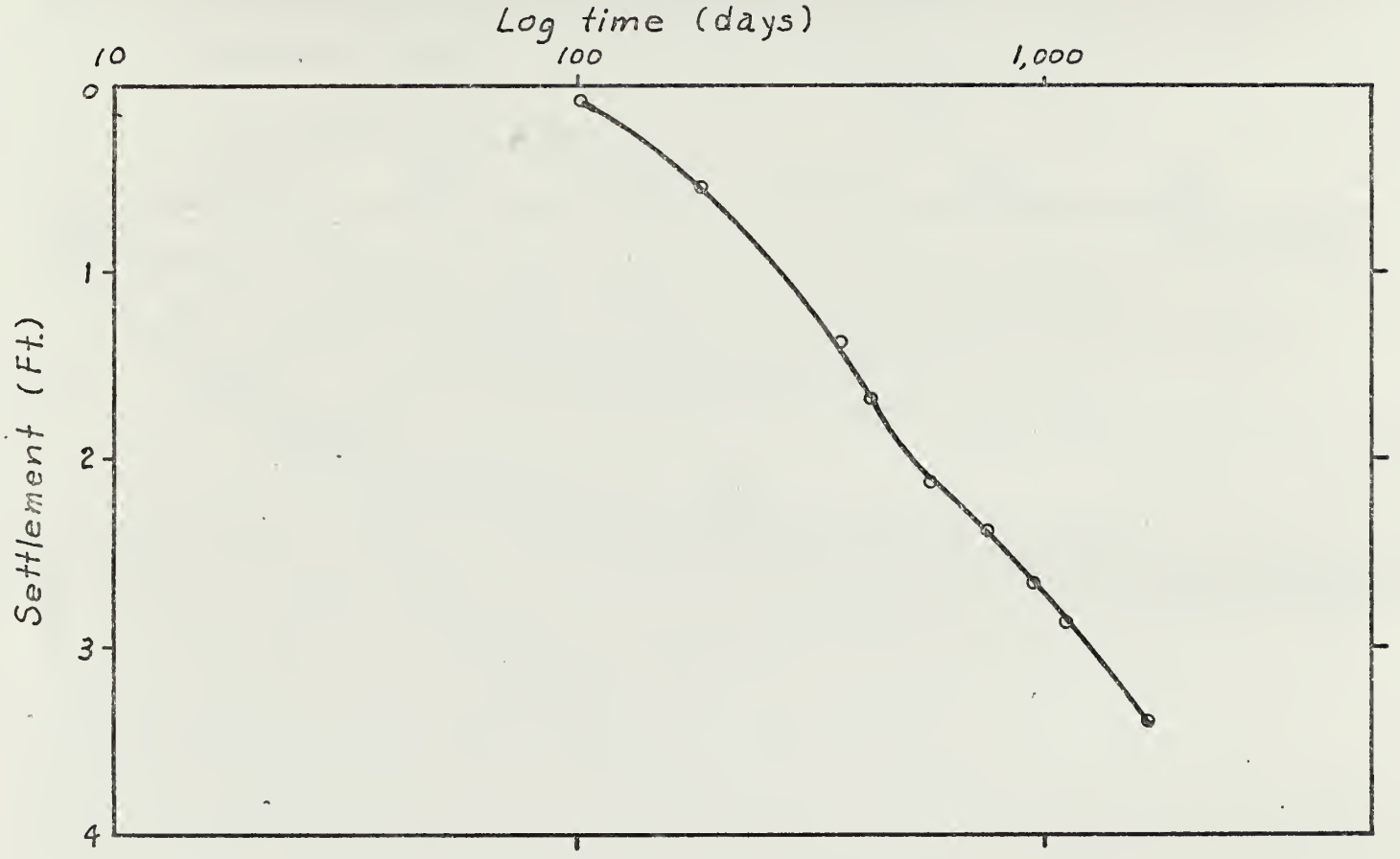


Figure 3-19

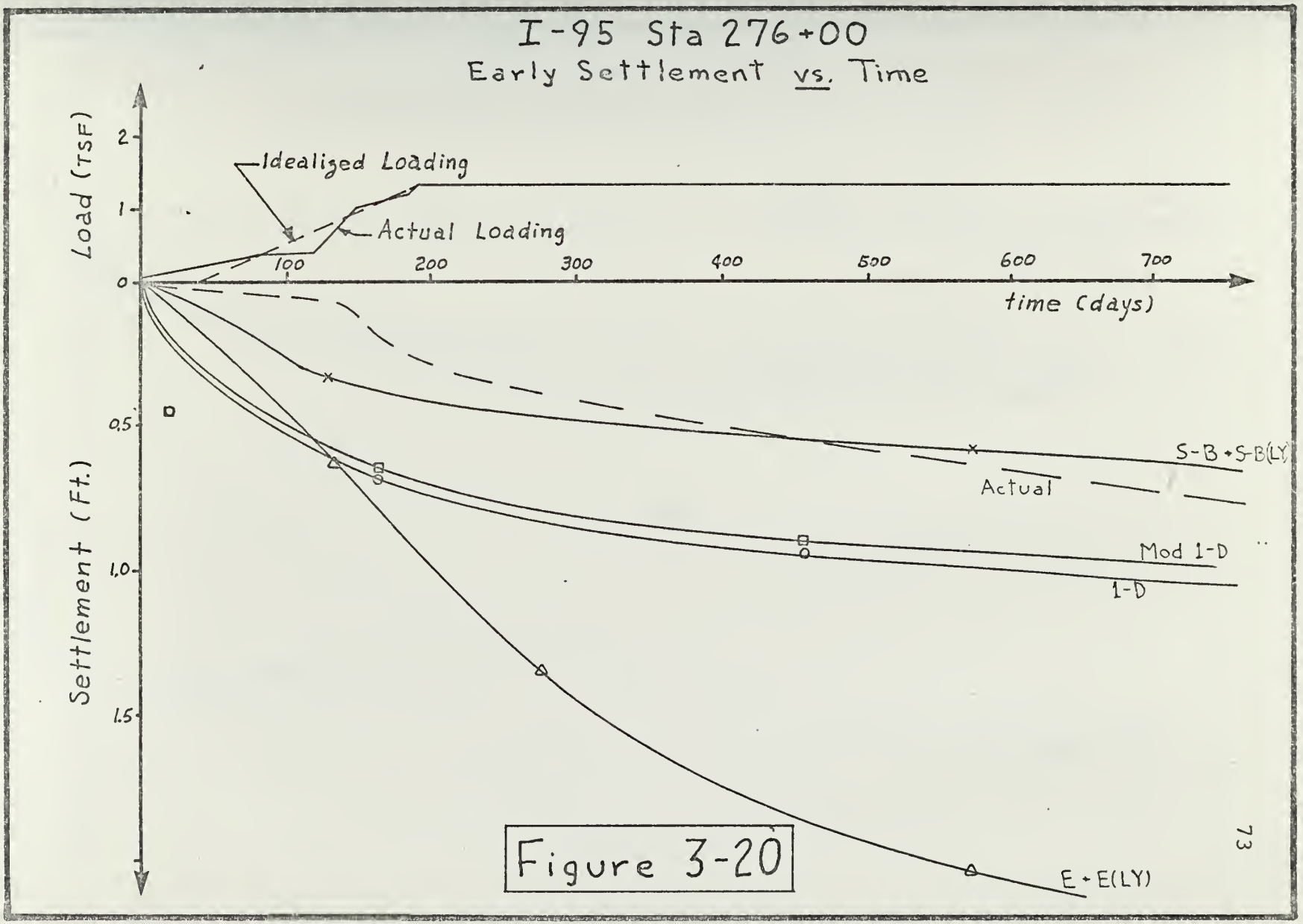

I-95 Sta 276+00
Early Settlement vs. Time
Load (TSF)
2
1
0
Idealized Loading
Actual Loading
100
200
300
400
500
600
700
time (days)
Settlement (Ft.)
0.5
1.0
1.5
S-B + S-B(LY)
Actual
Mod 1-D
1-D
E + E(LY)

Figure 3-20

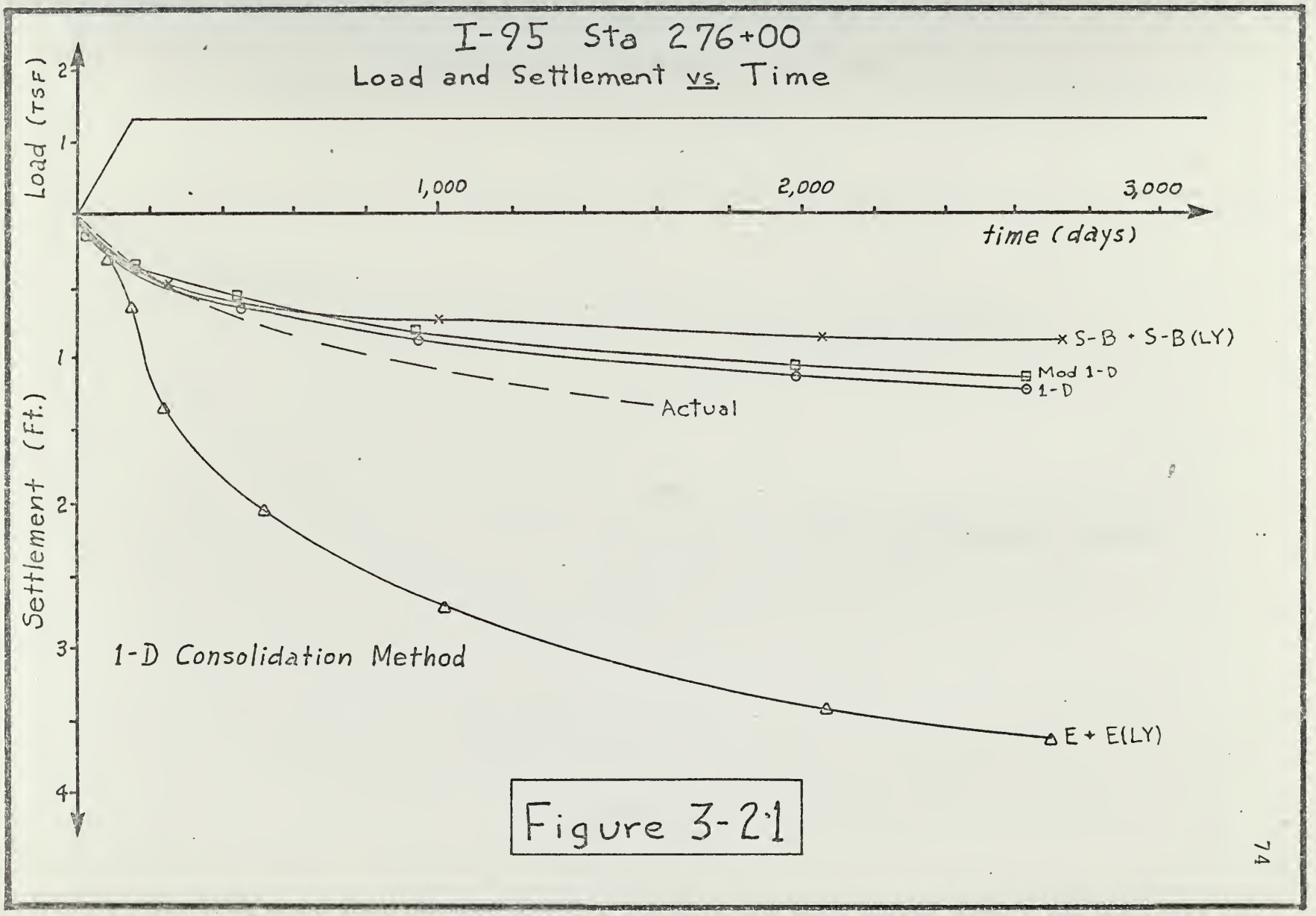

I-95 Sta 276+00
Load and Settlement vs. Time
Load (TSF)
1
2
1,000
2,000
3,000
time (days)
S-B + S-B (LY)
Mod 1-D
1-D
Actual
Settlement (Ft.)
1
2
3
4
1-D Consolidation Method
E + E(LY)

Figure 3-21

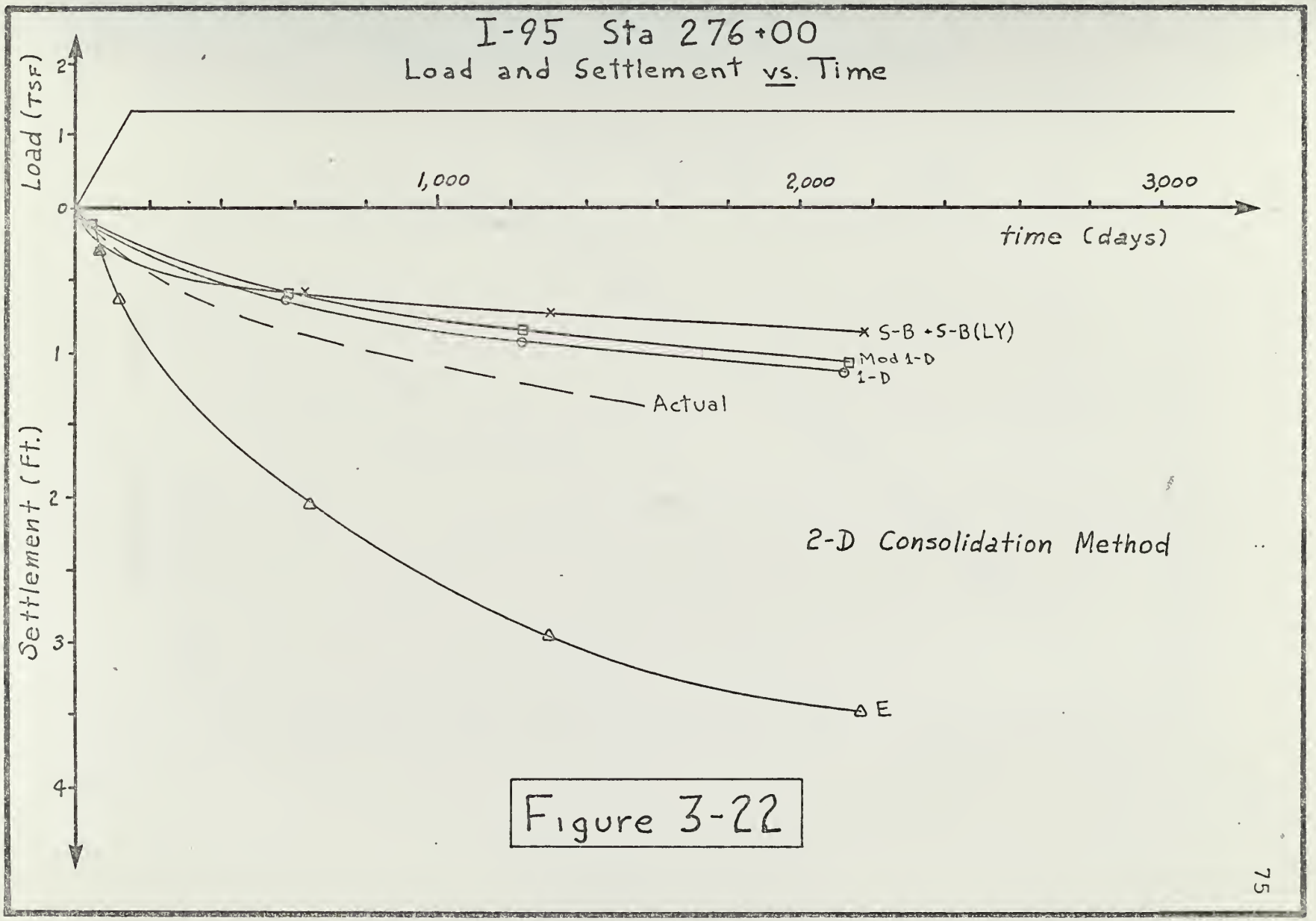

I-95 Sta 276+00
Load and Settlement vs. Time
Load (TSF)
2
1
0
1,000
2,000
3,000
time (days)
S-B + S-B(LY)
Mod 1-D
1-D
Actual
Settlement (Ft.)
1
2
3
4
E
2-D Consolidation Method

Figure 3-22

I-95 Sta 276+00
Measured Settlement vs. Log Time

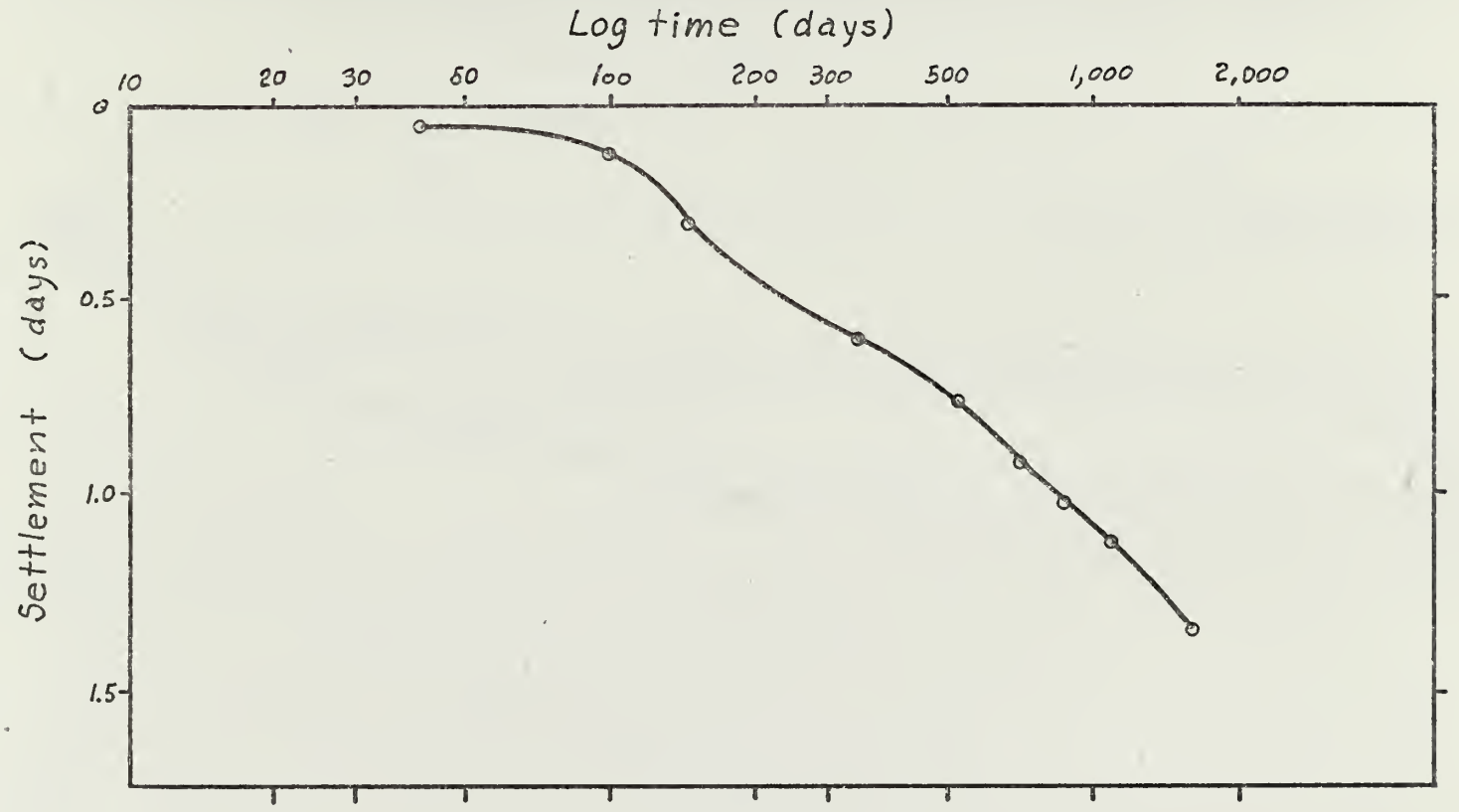


Figure 3-23

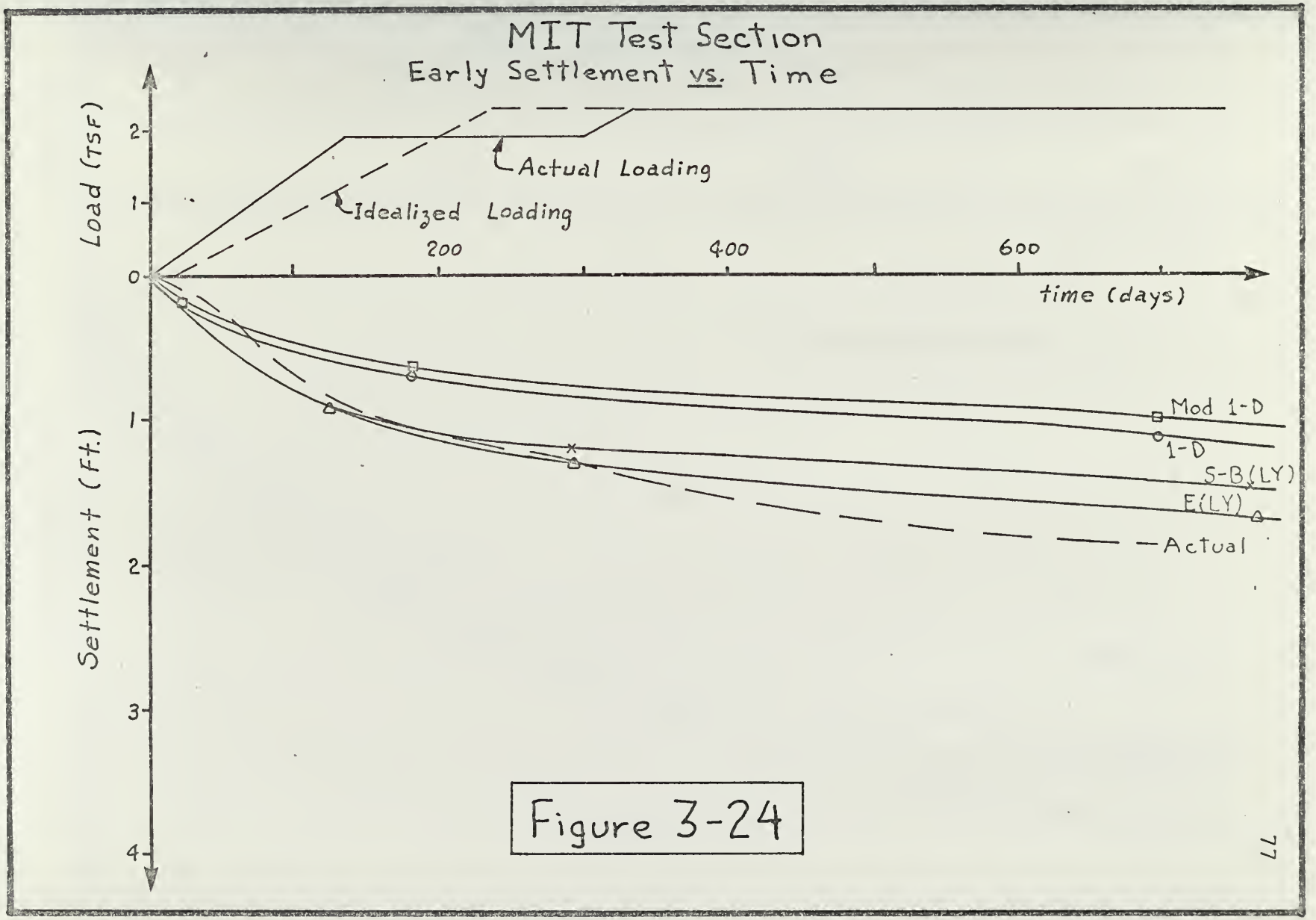

MIT Test Section
Early Settlement vs. Time
Load (TSF)
Settlement (Ft.)
2
1
0
1
2
3
4
200
400
600
time (days)
Actual Loading
Idealized Loading
Mod 1-D
1-D
S-B(LY)
E(LY)
Actual

Figure 3-24

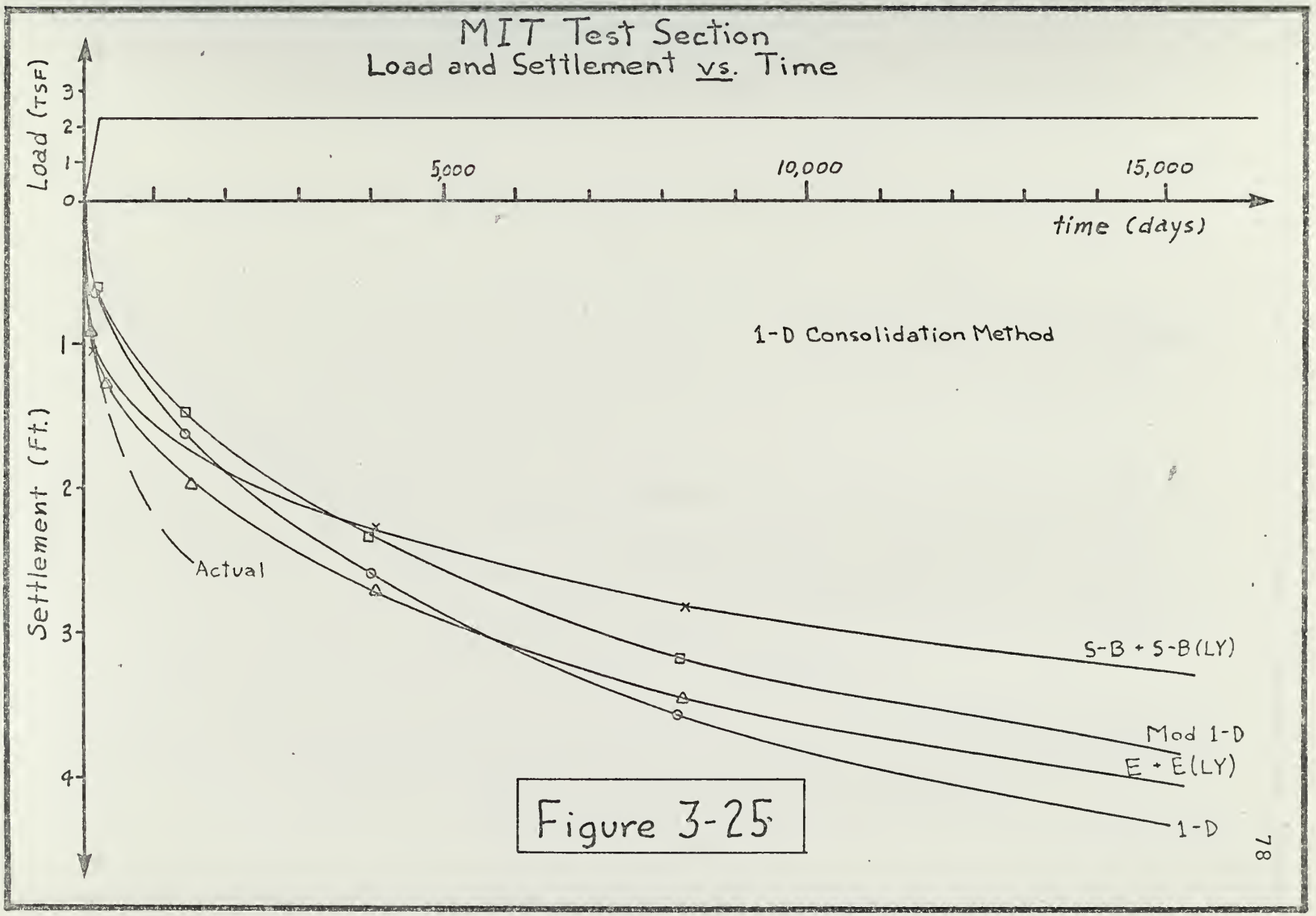

MIT Test Section
Load and Settlement vs. Time
Load (TSF)
0
1
2
3
5,000
10,000
15,000
time (days)
1-D Consolidation Method
Settlement (Ft.)
1
2
3
4
Actual
S-B + S-B(LY)
Mod 1-D
E + E(LY)
1-D

Figure 3-25

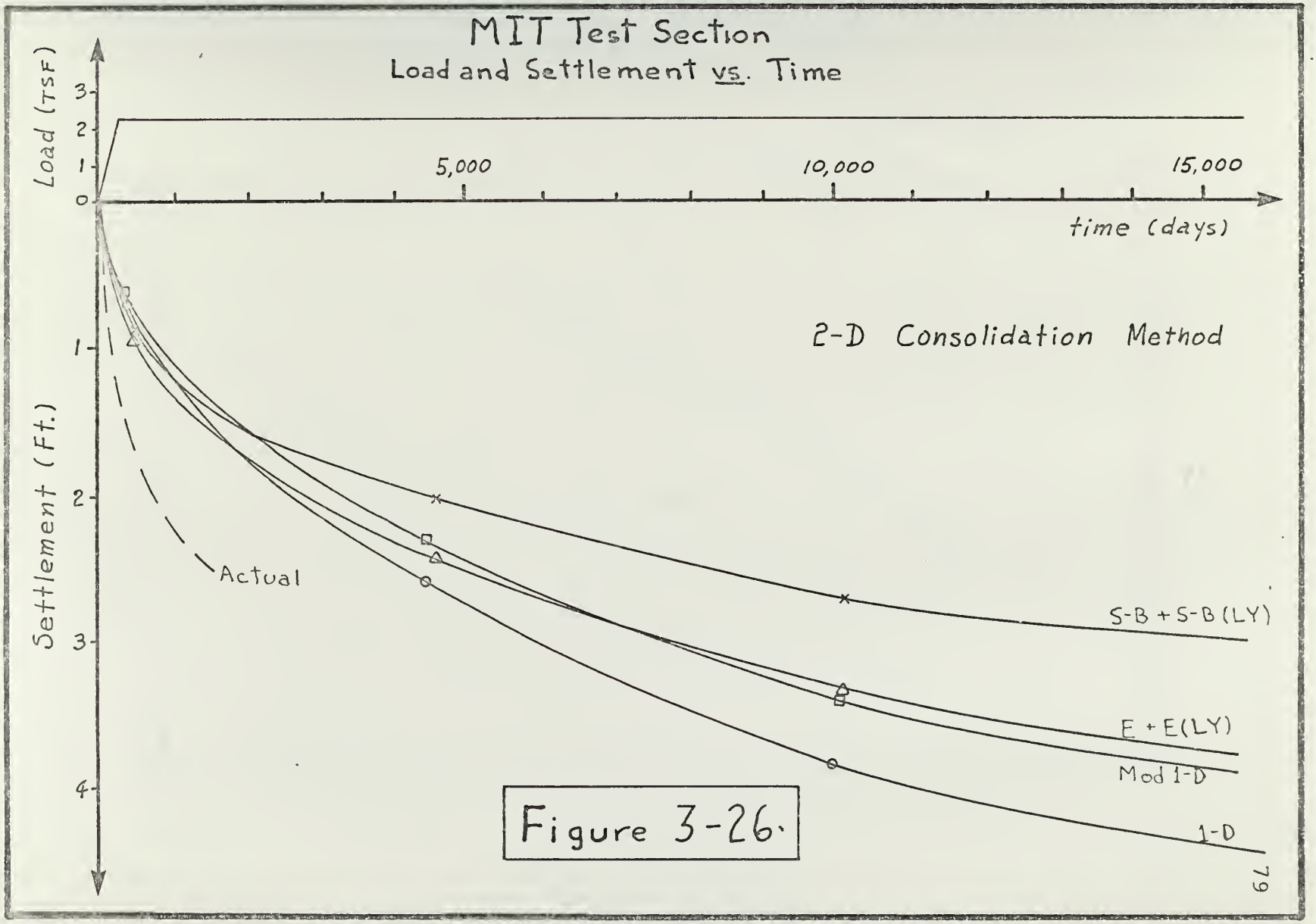

MIT Test Section
Load and Settlement vs. Time
Load (TSF)
0
1
2
3
5,000
10,000
15,000
time (days)
2-D Consolidation Method
Settlement (Ft.)
1
2
3
4
Actual
S-B + S-B (LY)
E + E(LY)
Mod 1-D
1-D
79

Figure 3-26.

MIT Test Section
Measured Settlement vs. Log Time

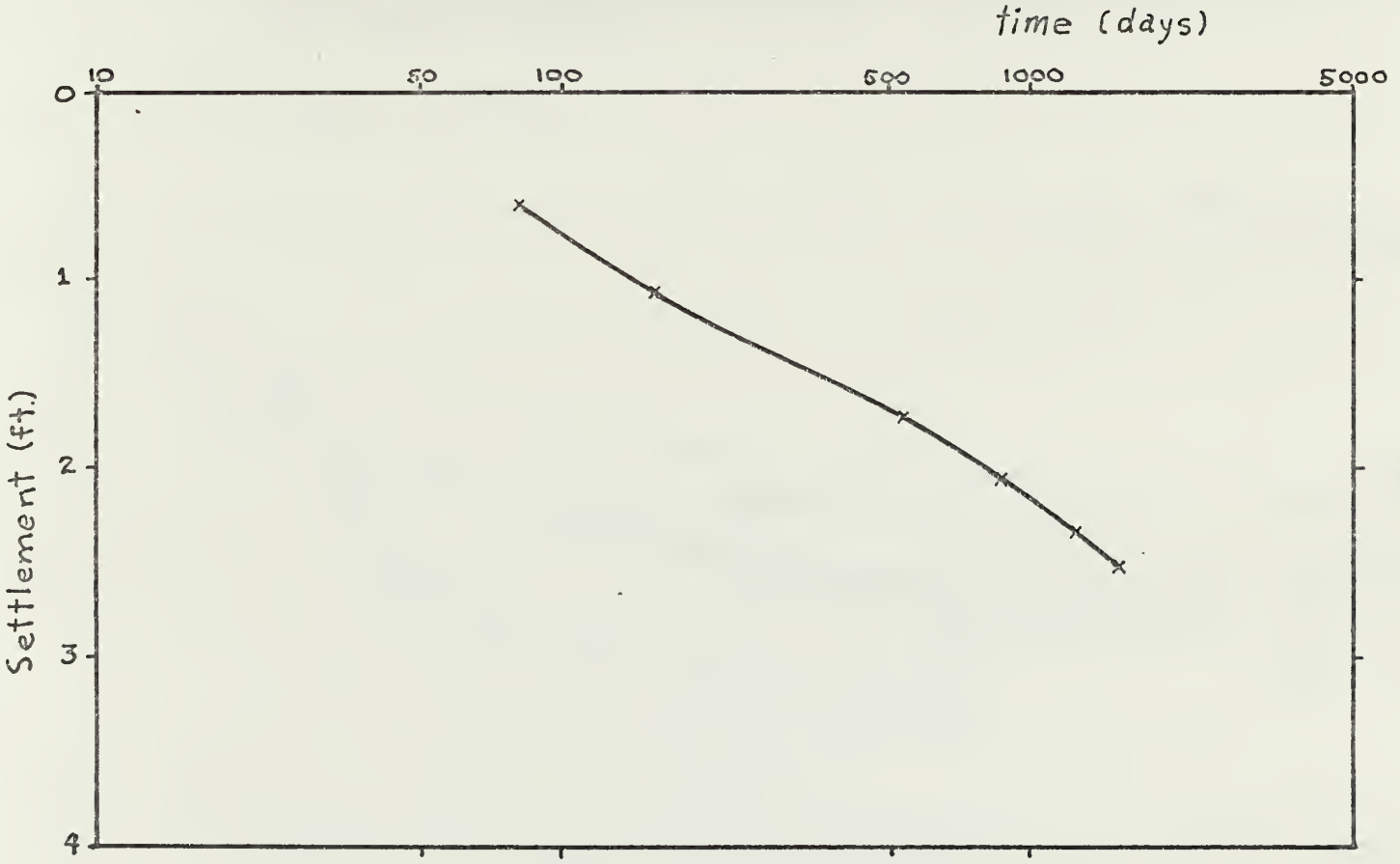


Figure 3-27

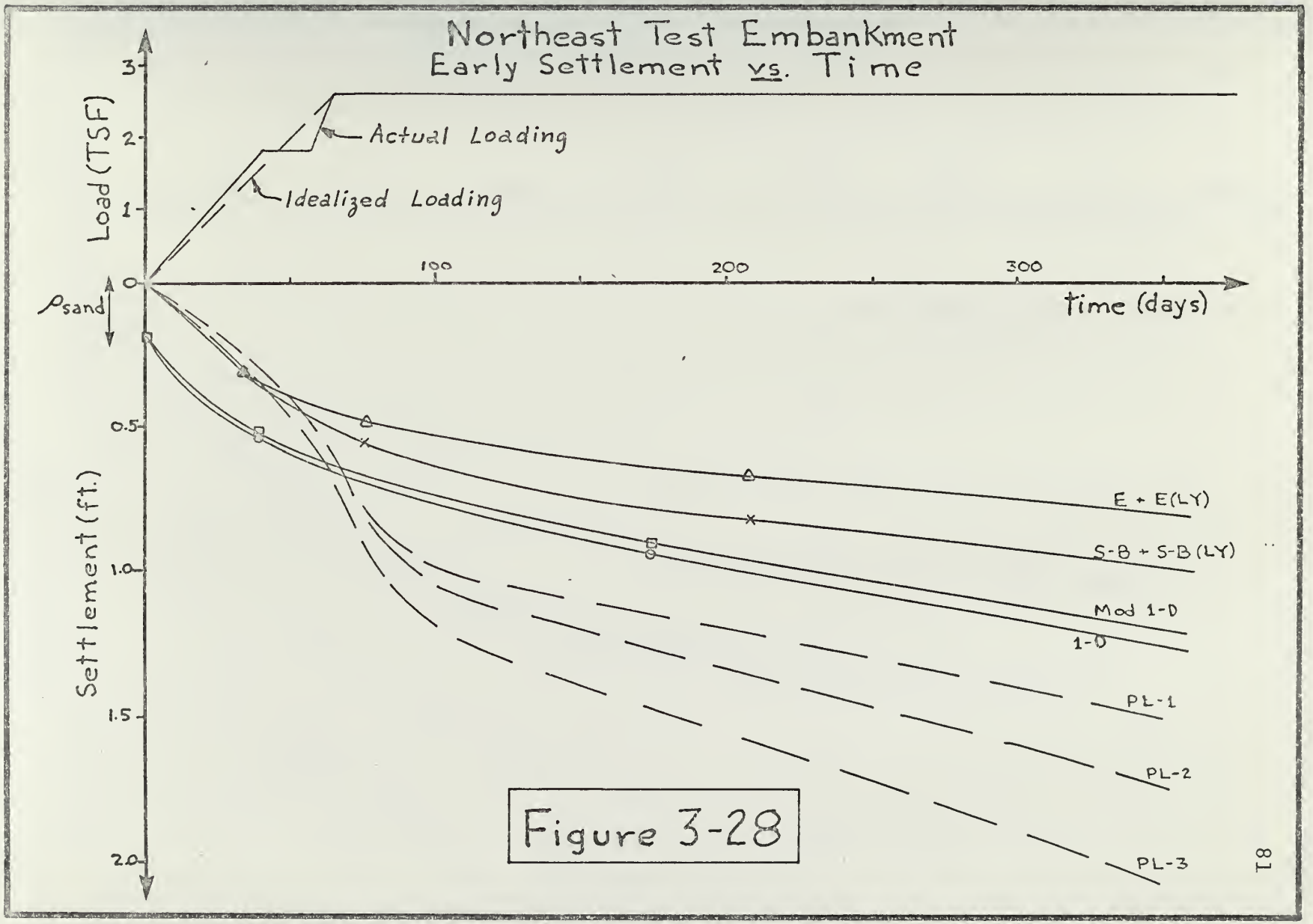

Northeast Test Embankment
Early Settlement vs. Time
Load (TSF)
3
2
1
0
Actual Loading
Idealized Loading
100
200
300
time (days)
ρsand
Settlement (ft.)
0.5
1.0
1.5
2.0
E + E(LY)
S-B + S-B(LY)
Mod 1-D
1-D
PL-1
PL-2
PL-3

Figure 3-28

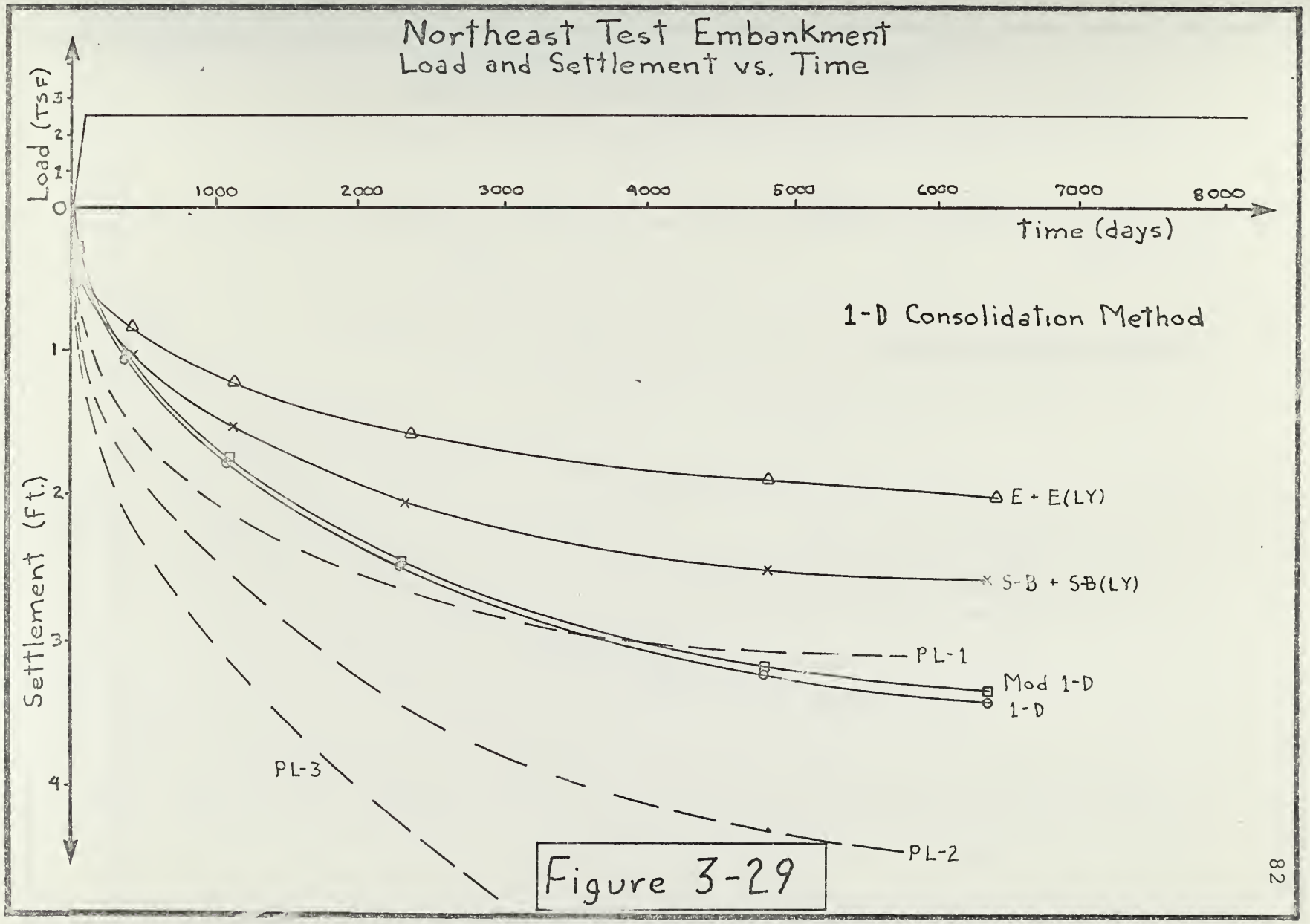

Northeast Test Embankment
Load and Settlement vs. Time
Load (TSF)
0
1
2
4
1000
2000
3000
4000
5000
6000
7000
8000
time (days)
1-D Consolidation Method
Settlement (Ft.)
1
2
3
4
E + E(LY)
S-B + S-B(LY)
PL-1
Mod 1-D
1-D
PL-3
PL-2

Figure 3-29

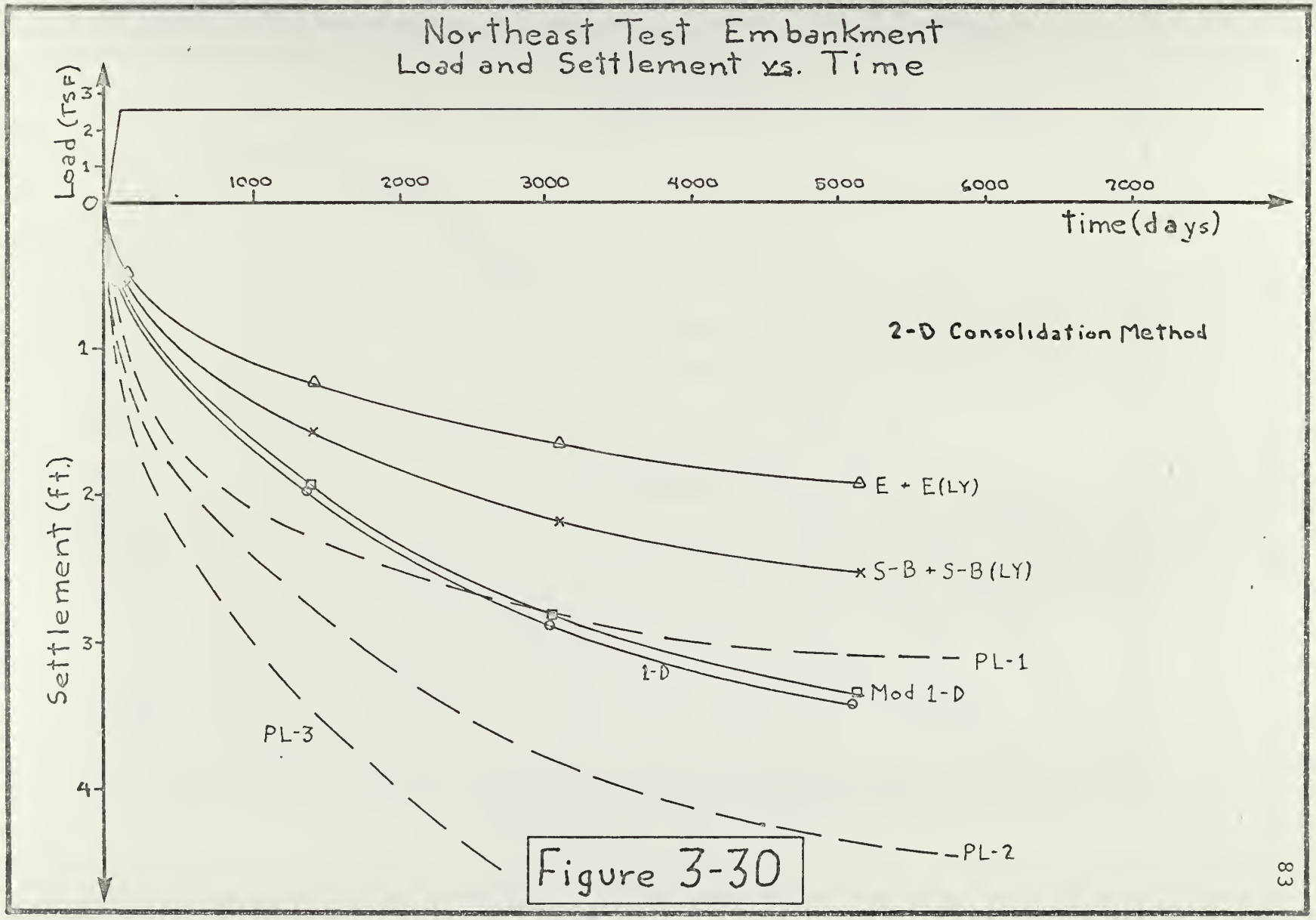

Northeast Test Embankment
Load and Settlement vs. Time
Load (TSF)
0
1
2
3
1000
2000
3000
4000
5000
6000
7000
time(days)
2-D Consolidation Method
Settlement (ft.)
1
2
3
4
E + E(LY)
S-B + S-B (LY)
PL-1
1-D
Mod 1-D
PL-3
PL-2

Figure 3-30

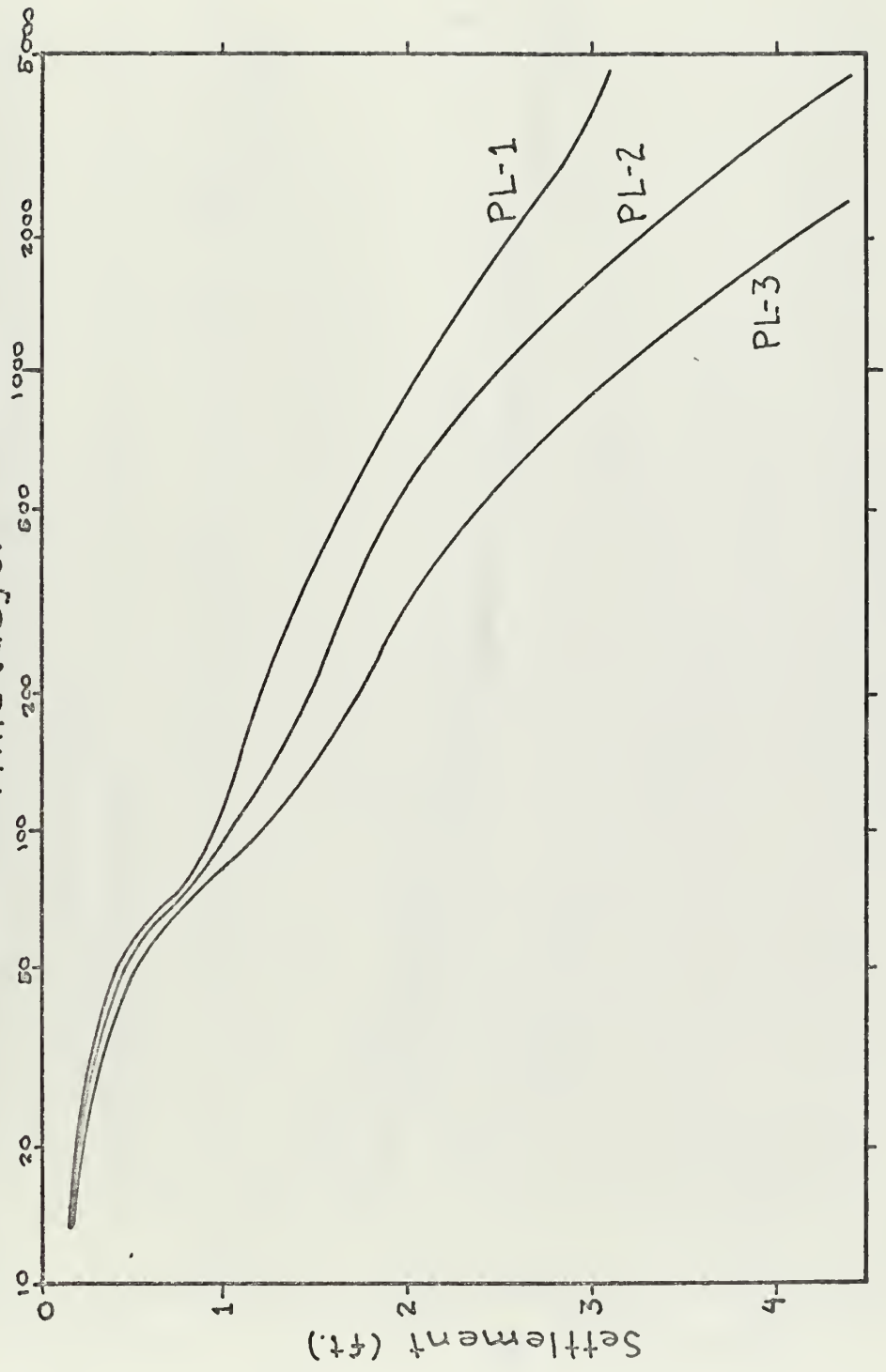

Northeast Test Embankment
Measured Settlement vs. Log Time
Time (days)
10
20
50
100
200
600
1000
2000
5000
0
1
2
3
4
Settlement (ft.)
PL-1
PL-2
PL-3

Figure 3-31

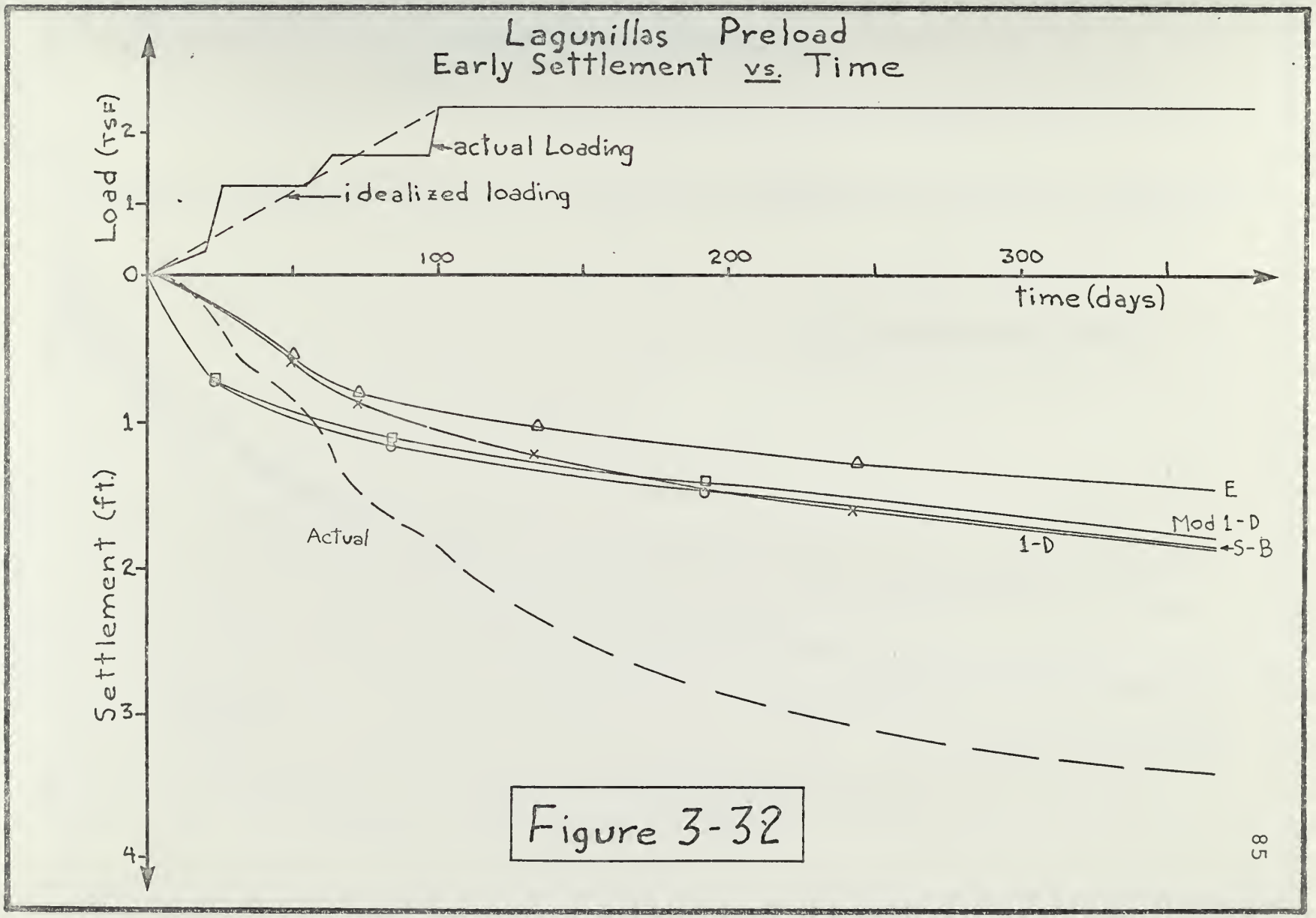

Lagunillas Preload
Early Settlement vs. Time
Load (TSF)
2
1
0
actual Loading
idealized loading
100
200
300
time (days)
Settlement (ft.)
1
2
3
4
Actual
E
Mod 1-D
1-D
S-B

Figure 3-32

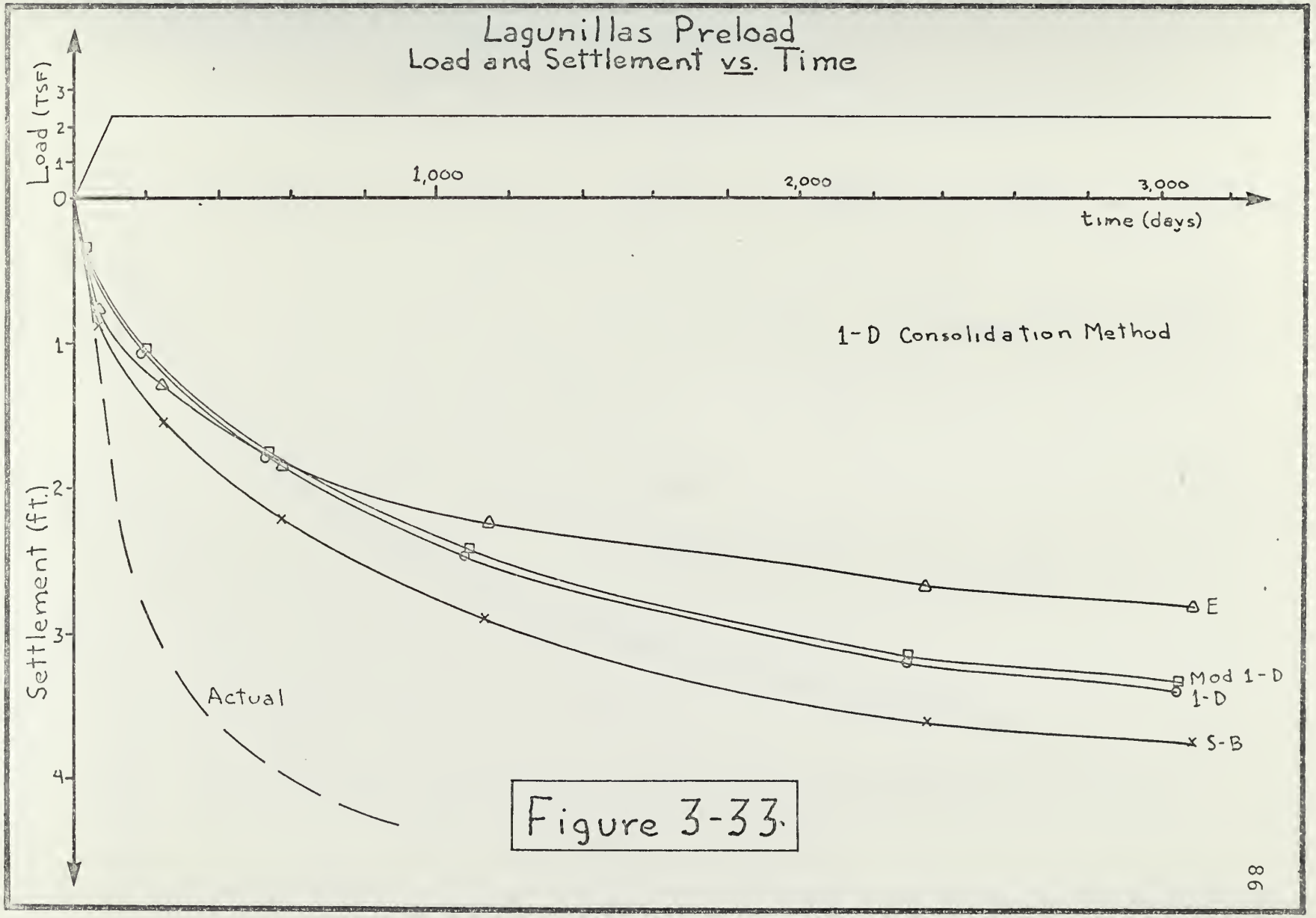

Lagunillas Preload
Load and Settlement vs. Time
Load (TSF)
0
1
2
3
1,000
2,000
3,000
time (days)
1-D Consolidation Method
Settlement (ft.)
1
2
3
4
Actual
E
Mod 1-D
1-D
S-B

Figure 3-33.

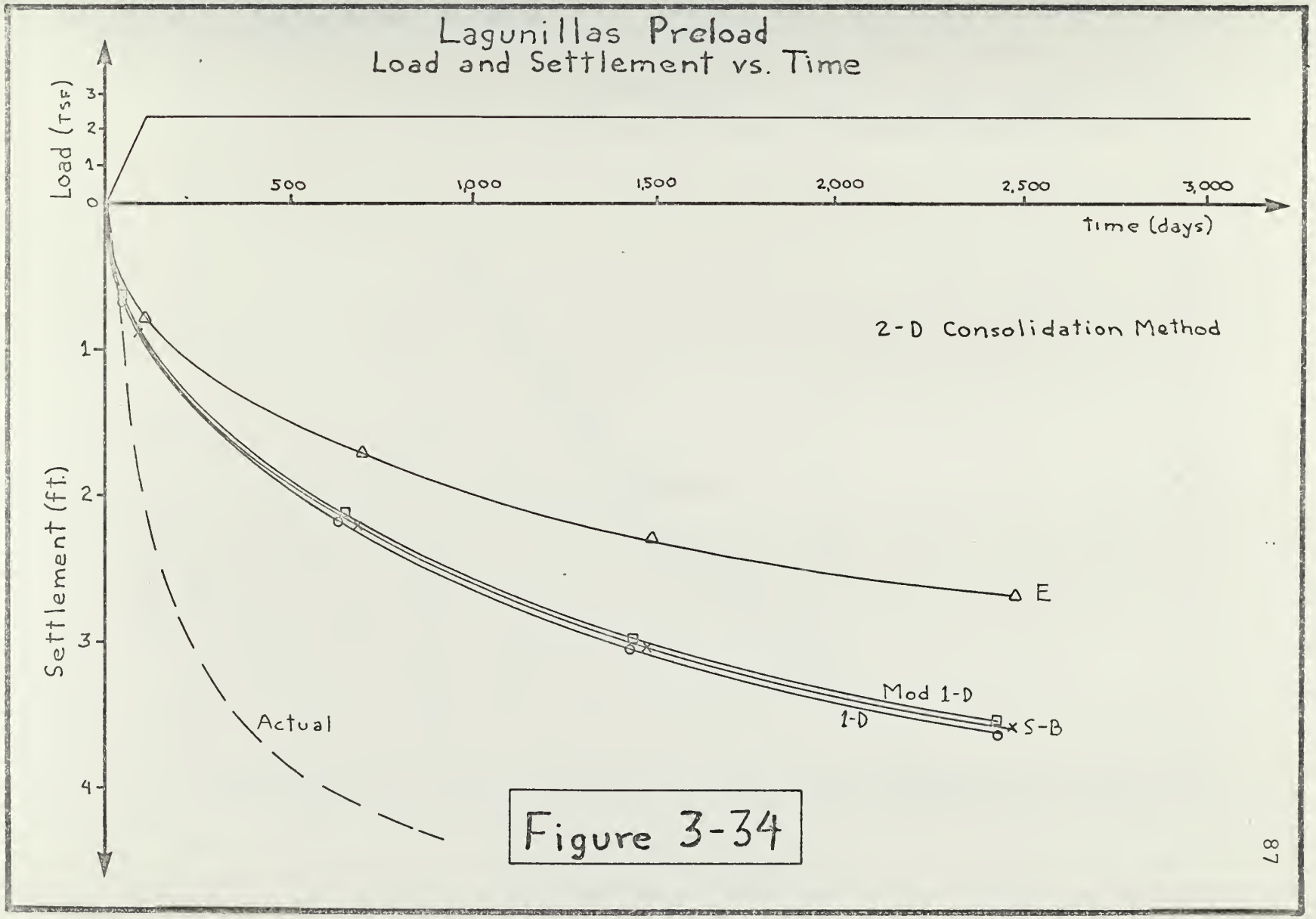

Lagunillas Preload
Load and Settlement vs. Time
Load (TSF)
0
1
2
3
500
1,000
1,500
2,000
2,500
3,000
time (days)
2-D Consolidation Method
Settlement (ft.)
1
2
3
4
E
Mod 1-D
1-D
S-B
Actual

Figure 3-34

Lagunillas Preload - Settlement vs. Log Time

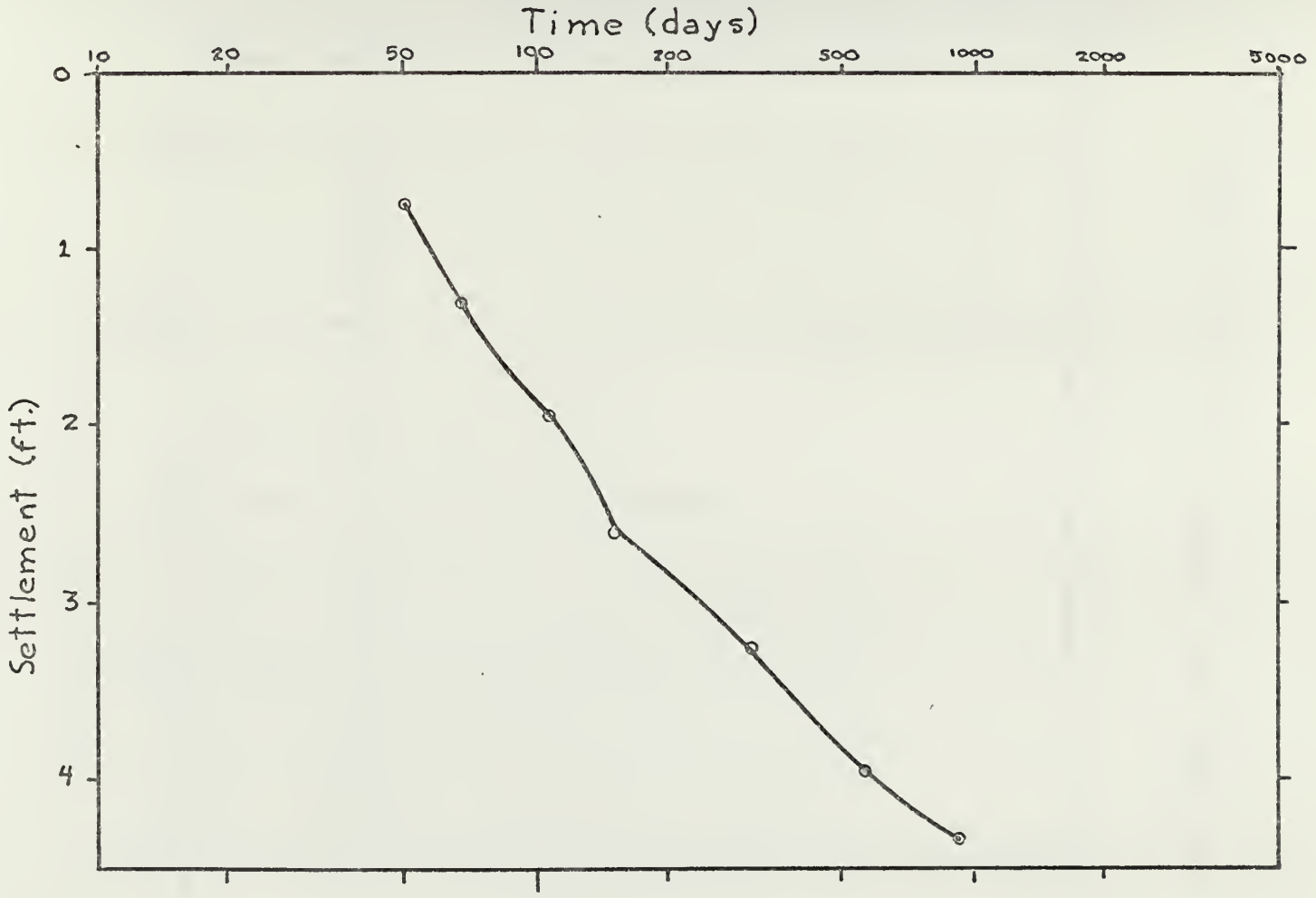


Figure 3-35

Effect of Parameters on Results

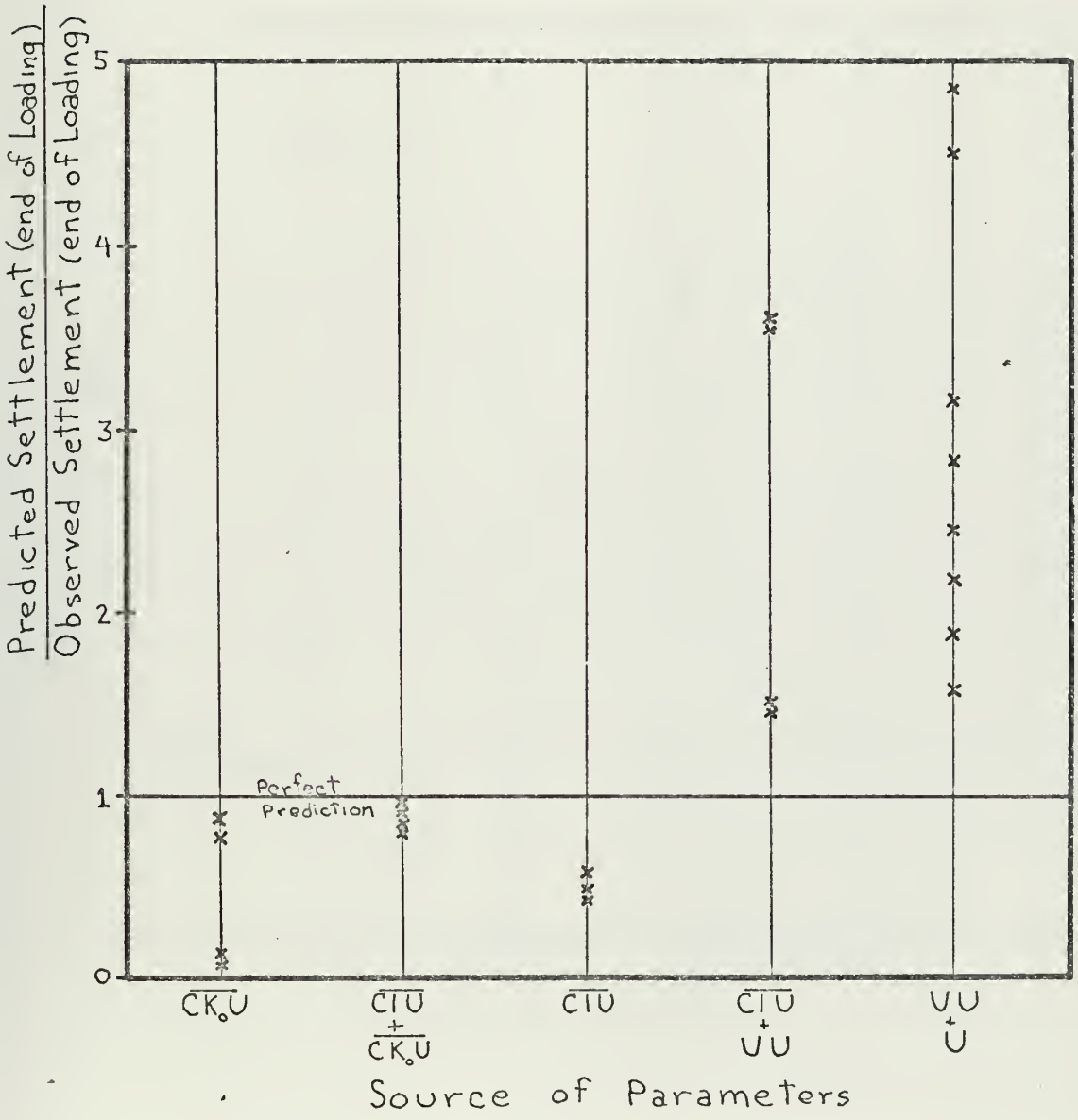


Figure 3-36

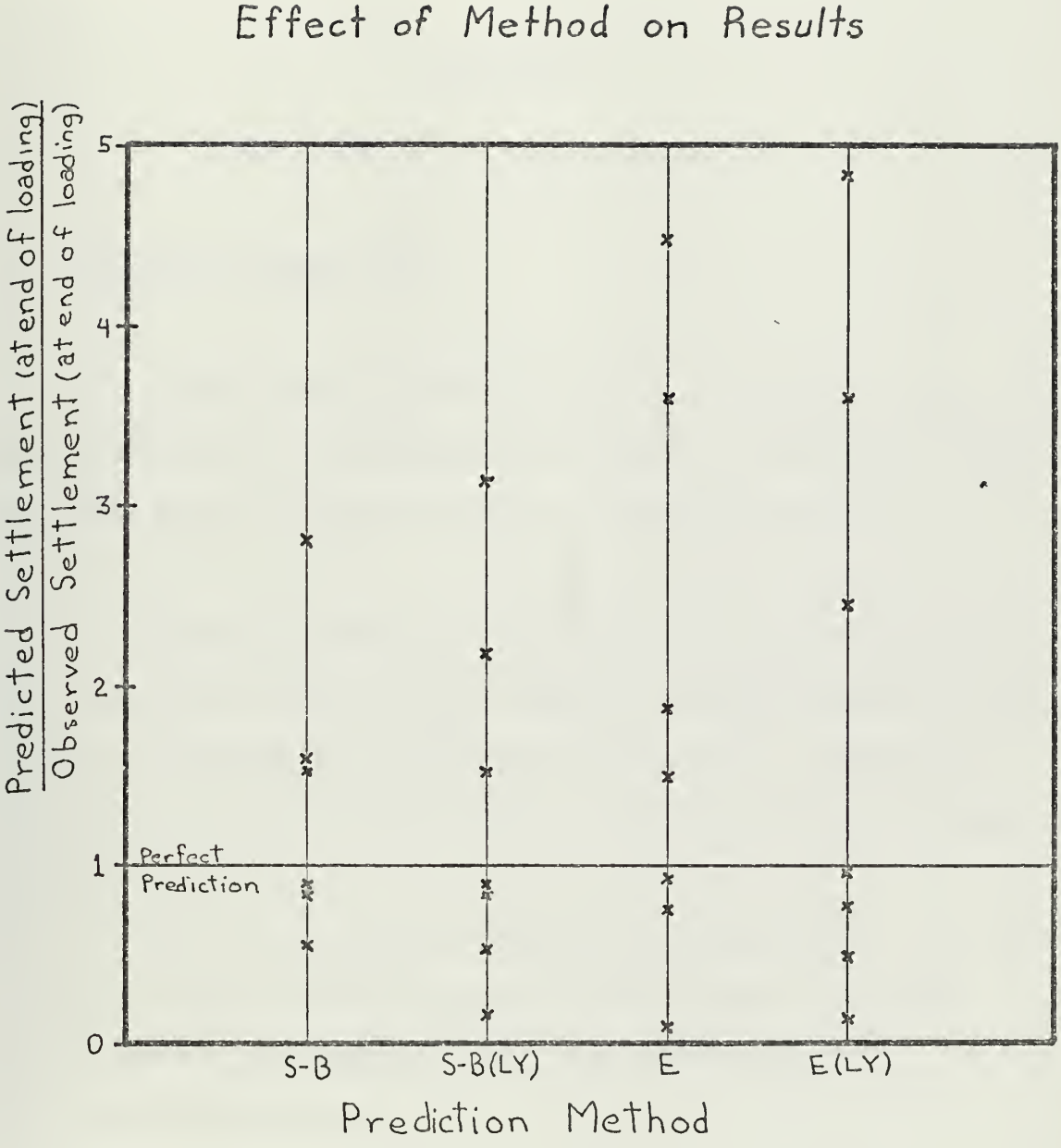

Effect of Method on Results
Predicted Settlement (at end of loading) / Observed Settlement (at end of loading)
5
4
3
2
1
0
Perfect Prediction
S-B
S-B(LY)
E
E(LY)
Prediction Method

Figure 3-37

APPENDIX A

MODELLING OF THE FIELD SITUATION

A.1 SUBSURFACE MODELLING

1. First, obtain borings and soil classifications. They should be spaced at an adequate distance to determine accurately the location and variation of layers and lenses of soil.

2. From the boring logs, draw the model of the profile. When the layers slope, take an average thickness of the layer and make all boundaries horizontal. (When possible, try to use borings which are within 250 feet of the point in question.)

3. For a very deep soft layer (depth > 5B):

   a. for the elastic method, solutions are available in Lysmer and Duncan (1972) for semi-infinite elastic half space;

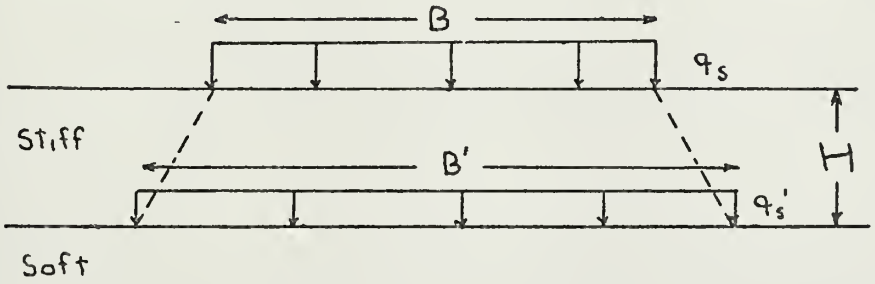
   b. for all other methods, consider a rigid base located at depth = 5B.

4. For a soft layer overlying a very thick stiff layer, consider the stiff layer to be rigid base. (Till is considered to be a rigid base.)

5. For a thin stiff layer overlying a soft layer:

a. for all methods except 1-D and modified 1-D

(1) consider the stiff layer as transmitting the force as shown below



(2) idealize the system such that there is just a soft layer loaded in the same manner as the actual case but with a modified load width ($B'=B+1.096H$) and a modified load intensity ($q_s=\frac{q_s B}{B'}$).

b. For 1-D and Modified 1-D Methods assume the stiff layer does not deform but transmits stress undiminished to the soft layer below.

A.2 MODELLING THE LOAD SHAPE

1. Circular loads should be treated as such for all methods.

2. Square and rectangular loads:

a. for the elastic method use actual shape for model;

b. for the 1-D and Modified 1-D Method model as a one dimensional circular load;

c. for the Skempton-Bjerrum Method treat this load as a square or rectangle when computing initial settlement (ρ_i) and as a one dimensional circular load for consolidation settlement (ρ_c).

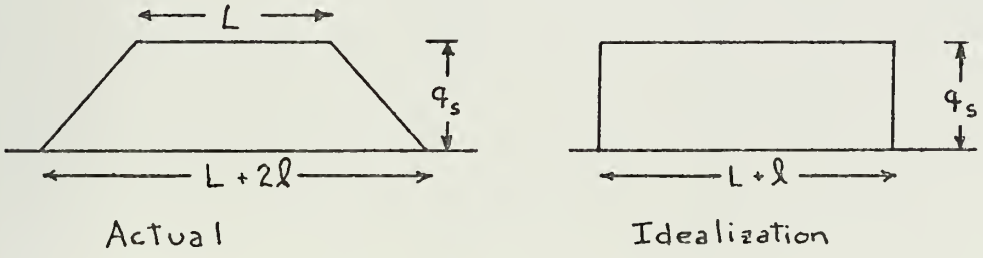
3. Strip Loads:

a. for 1-D and Modified 1-D Methods treat the load as a one dimensional circular load;

b. for all other methods use the actual strip load dimensions.

4. Embankment Shaped Loads:

a. For all methods idealize the embankment into a strip load as shown below and follow the directions of the strip load above.



A.3 MODELLING THE LOADING TIME PERIOD

1. For One Dimensional and Modified One Dimensional Methods, consider the load as being instantaneously applied.

2. For Elastic and Skempton-Bjerrum Methods use the techniques described by Taylor (1948). Technique is summarized in Appendix C Section C.2 .

APPENDIX B

TOTAL SETTLEMENT-METHODS, PARAMETERS AND COMPUTATION TECHNIQUES

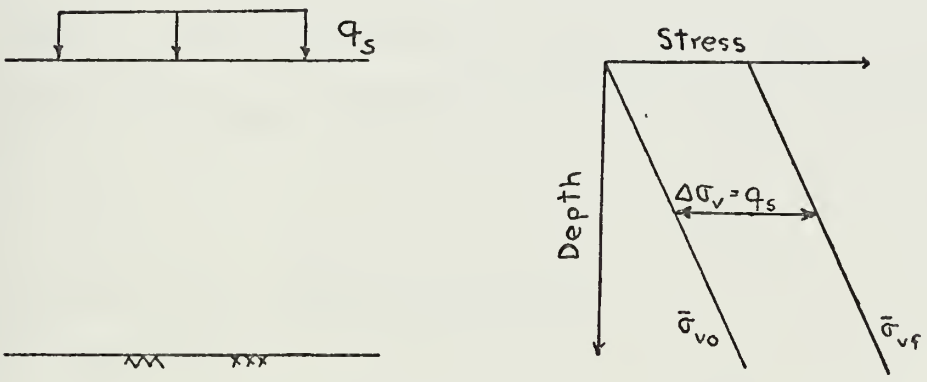
B.1 ONE-DIMENSIONAL METHOD

1. Stresses

$\bar{\sigma}_{vo}$ - from Lambe and Whitman (1969)

$\bar{\sigma}_{vf} = \sigma_{vo} + \sigma_v$

$\Delta\sigma_v = q_s$ = Applied Load



2. Parameters

$\bar{\sigma}_{vo}$ - as shown above

$\bar{\sigma}_{vf}$ - as shown above

$\bar{\sigma}_{vm}$ - from oedometer tests using Schmertmann's Method (Ladd, 1971)

$CR = \frac{C_c}{1+C_o}$ - from oedometer test "field curve", a weight average over the depth

H_i = height of the i^{th} layer

3. Computation Techniques

   a. divide the soil into natural layers then subdivide to get at least five (5) layers.

   b. Compute the total settlement (ρ_t) by summing the contribution from each layer:

$$\rho_{TF} = \Sigma \; H_i \left(RR \log \frac{\bar{\sigma}_{vm}}{\bar{\sigma}_{vo}} + CR \log \frac{\bar{\sigma}_{vf}}{\bar{\sigma}_{vm}}\right) + \rho_{SAND}$$

Where ρ_{SAND} is the settlement of the sand layer(s), if any. Computations of ρ_{SAND} is explained in Section B.6 of this Appendix.

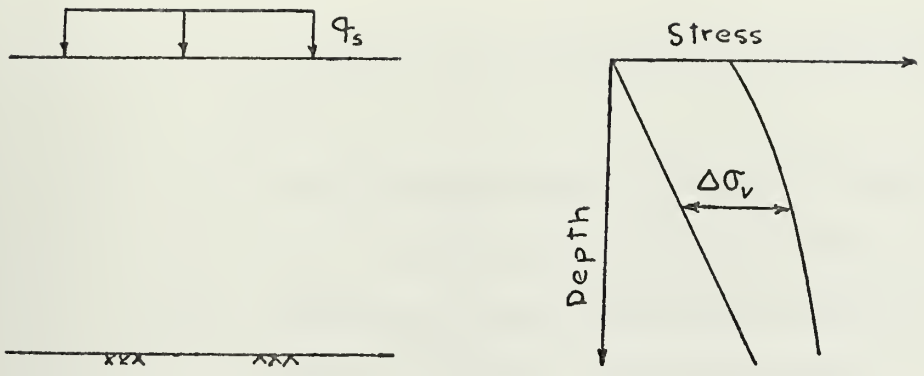
B.2 MODIFIED ONE-DIMENSIONAL METHOD

1. Stresses

   $\bar{\sigma}_{vo}$ - from Lambe and Whitman (1969)

   $\bar{\sigma}_{vf} = \bar{\sigma}_{vo} + \Delta\sigma_v$

   $\Delta\sigma_v$ - stress increment from an elastic theory solution as tabulated by Lysmer and Duncan (1972).



2. Parameters

$\bar{\sigma}_{vo}$ - as shown above

$\bar{\sigma}_{vf}$ - as shown above

$\bar{\sigma}_{vm}$ - determined by exactly the same means as the One-Dimensional Method

H_i - same as One-Dimensional Method

CR - same as One-Dimensional Method

RR - same as One-Dimensional Method

3. Computation technqiues are the same as the One-Dimensional Method.

B.3 ELASTIC METHOD

1. Stress are the same as those computed for the Modified One-Dimensional Method.

2. Parameters

B - load width (or diameter) from the model.

H - thickness of compressible stratum from the model.

μ_i = 0.50 the undrained Poisson's ratio.

μ_c = 0.30 the drained Poisson's ratio.

E_i - undrained Young's Modulus defined at one half the maximum deviator stress.

E_i determined by one of the following methods:

$\overline{CK_oU}C$ Most Preferable

$\overline{CIU}C$ ↓

UU

U Least Preferable

EMPIRICAL METHOD

E_c - drained Young's Modulus defined at one half the maximum deviator stress.

E_c determined by one of the following methods:

$\overline{CK_oD}C$ Most Preferable

$\overline{CID}C$ ↓

Emperical (E_c=1/3E_i) Least Preferable

3. Computation Techniques

a. determine moduli (E_i and E_c) throughout depth of soil stratum.

b. take weighted average of moduli over depth (include sand layers for E_i).

c. enter elastic solution charts in Lysmer and Duncan (1972) with H/B and μ to get $I_{\Delta Z}$ for both drained and undrained

d. Compute:

Initial Elastic Settlement $\rho_e = \frac{q_s H}{\Pi E_i} I_{\Delta Z_i}$

Final Consolidation Settlement $\rho_{CF} = \frac{q_s H}{\Pi E_c} I_{\Delta Z_C}$

Final Total Settlement $\rho_{TF} = \rho_e + \rho_{CF}$

B.4 ELASTIC METHOD CONSIDERING FIRST LOCAL YIELD

1. Stress are the same as those for the Modified One Dimensional Method.

2. Paramters

ρ_e - from the previous method

ρ_{CF} - from the previous method

ϕ - from the previous method

PI versus depth - from index properties

OCR versus depth - from oedometer tests

K_o versus depth from modified oedometer and OCR

or empirical: normal consolidation clays

$K_o = 0.95 - \text{Sin } \bar{\phi}$ fig 1-1, Ladd (1971)

s_u versus depth - from results of one of the following tests

Most Preferable	$\overline{CK_oU}$ Direct Simple Shear
↓	Average of $\overline{CK_oUC}$ and $\overline{CK_oUE}$ Triaxial
↓	$\overline{CK_oUE}$ Triaxial
↓	$\overline{CK_oUC}$ Triaxial
↓	$\overline{CIUC}$ Triaxial
↓	FV for sensitive soils
Least Preferable	UU for plastic soils

3. Computation Technqiues

   a. Compute ρ_e and ρ_{CF} from the previous method

   b. Determine ultimate bearing capacity (q_{ult}) as prescribed by Lambe and Whitman (1969).

   c. Determine the Applied Stress Ratio $= \frac{q_s}{q_{ult}}$

   d. Determine the Initial Shear Stress Ratio (f):

   $f = \frac{1-K_o}{2\ s_u/\sigma_{vo}}$ where K_o, s_u, $\bar{\sigma}_{vo}$ are averages weighted over the depth.

   e. Refer to D'Appolonia, et al. (1971), Figure 3 to get Settlement Ratio (S_R).

   f. Calculate Initial Settlement (ρ_i)

$$\rho_i = \frac{\rho_e}{S_R}$$

g. Calculate Final Total Settlement

$$\rho_{TF} = \rho_i + \rho_{CF}$$

B.5 SKEMPTON-BJERRUM METHOD

1. Stress are the same as those for Modified One Dimensional Method.

2. Parameters

ρ_i - initial settlement from previous calculations

$\rho_{OED} = \rho_{CF}$ from Modified One-Dimensional Method

A - Skempton's A parameter from triaxial tests:

defined at one half the strain at failure.

$\overline{CK_oUC}$ Preferable

$\overline{CIUC}$

$$A = \frac{\Delta U - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}$$

H/B - from model

α an empirical constant based on geometry:

	α	
H/B	Circular Load	Strip Load
0	1.0	1.0
0.25	0.67	0.74
0.50	0.50	0.53
1.0	0.38	0.37
2.0	0.30	0.26
4.0	0.28	0.20
10.0	0.26	0.14
∞	0.25	0

3. Computation Techniques

   a. Divide soil up into layers and find ρ_{OED} (= ρ_{CF} by modified One Dimensional Method) for each layer.

   b. Plot A versus depth and find A for the mid-point of each layer.

   c. Determine the Skempton Bjerrum factor, μ', for each layer:

      (1) when the A parameter is known

      $\mu' = A + \alpha(1-A)$

      (2) when the A parameter is not known determine it from this empirical chart:

Average OCR	μ'
1.00-1.29	0.85
1.30-1.69	0.70
1.70-2.59	0.60
2.60-4.00	0.50
>4.00	0.40
Sensative Clays μ'=1.20	

   d. Calculate the final consolidation settlement (ρ_{CF}); $\rho_{CF} = \mu' \rho_{OED}$

e. Calculate the final total settlement

$$\rho_{TF} = \rho_i + \rho_{CF}$$

B.6 SETTLEMENT DUE SAND LAYERS FOR ONE-DIMENSIONAL AND MODIFIED ONE-DIMENSIONAL METHODS.

1. Stress increment ($\Delta\sigma_v$) should be determined by the appropriate method under consideration.

2. Parameters

E_s - Young's Modulus for the sand from one of the following means:

$\overline{CK_oDC}$	Most Preferable
$\overline{CIDC}$	↓
Empirical	Least Preferable

By the empirical means - $E_s = 2q_c$ where q_c = static cone resistance (TSF) or by the following (Schmertmann, 1970):

Type of Soil	q_c/N	E_s (TSF)
Silts, Sandy silts Slighly Cohesive Silt - Sand Mix.	2.0	4.0 N
Clean, Fine to Med. Sands and Slighly Silty Sands	3.5	7.0 N
Coarse Sands and Sands with little Gravel	5.0	10.0 N
Sand Gravels and Gravels	6.0	12.0 N

where N = average Standard Penetration Blow Count (blows per foot); H_{sand} thickness of the sand layers.

$\Delta\sigma_v$ - the stress increment at the mid-point of the sand layer.

3. Computation Techniques

$$\rho_{SAND} = H_{SAND} \frac{\Delta\sigma_v}{E_s}$$

APPENDIX C

TIME RATE OF SETTLEMENT

C.1 CHOOSING A VALUE FOR C_v

1. Using results of oedometer tests at various depths, plot c_v (square root time and log time) versus log stress.

2. C_v will become about constant in the normally consolidated region. Fine the average of c_v (log time) and c_v (square root time) in this region this is c_{vn}. If only C_v (log time) is available then multiply this by 1.5 to get c_{vn}. If only c_v (square root time) is available then multiply this by 0.67 to get c_{vn}.

3. From the techniques described in the Modified One Dimensional Method (Section B.2 of Appendix B), determine for the various layers:

$$E_{vo} = RR \log \frac{\bar{\sigma}_{vm}}{\bar{\sigma}_{vo}} \quad \text{(strain due to recompression)}$$

$$E_{vn} = CR \log \frac{\bar{\sigma}_{vf}}{\bar{\sigma}_{vm}} \quad \text{(strain due to virgin compression)}$$

4. Compute the ratio $\frac{\varepsilon_{vo}}{\varepsilon_{vo} + \varepsilon_{vn}}$ for each layer.

5. Using the following equation, solve for c_v' .

$$\frac{c_v - c_{vn}}{9\ C_{vn}} = \frac{\varepsilon_{vo}}{\varepsilon_{vo} + \varepsilon_{vn}}$$

This is graphically presented in Figure C-1. c_v' is the appropriate value of the coefficient of consolidation for the depth of that layer. This equation is based on the assumption that c_v' in the overconsolidated region is ten (10) times that in the noramlly consolidated region. It is also based on the assumption that the true c_v' will be proportional to the relative percentages of strain due to recompression and due to virgin compression.

6. Plot c_v' versus depth.

7. Compare the values of c_v' over the depth. If the values of c_v' in one particular region are greater than the values of c_v' in another by a factor of 20 or more, than divide the soil into distinct layers and take a weighted average of the value of C_v' within each layer.

8. Use the precedure described by Leonards (1962) to make one equivalent thickness of soil and one value of c_v'. This procedure is demonstrated in Figure C-2. When there are N distinct layers (N>2) then follow the procedure shown in the figure N-1 times, progressively reducing the number of layers.

9. If c_v' does not vary by a factor of 20 or more over the depth, then merely take a weighted average.

C.2 ONE DIMENSIONAL CONSOLIDATION METHOD

1. Parameters

c_v - Coefficient of consolidation as described in Section C.1 of this appendix.

H_d - The drainage path as described in Lambe and Whitman (1969). If a transformed thickness H_t was calculated (Section C.1 of this appendix), then H_d will be a function of H_t.

$\bar{U}$ - The average degree of consolidation.

T - The time factor, related to U by Terzaghi consolidation theory as shown.

$\bar{U}$	0	10	20	30	40	50	60	70	80	90	95	100
T	0	.008	.031	.071	.126	.196	.287	.403	.567	.848	1.125	∞

2. Computation Techniques

a. Taylor (1948) demonstrated how to deal with gradual loading. Basically, consider the load as being applied instantaneously at the middle of the loading period. Initial settlement is assumed to occur between time "zero" and one half of the loading period. Consolidation settlement begins at the middle of the loading period. This is demonstrated in Figure C-3.

b. Compute $\rho_T = \rho_i + U\rho_{CF}$ for varying degrees of consolidation.

c. Compute the times which correspond to the varying degrees of consolidation (and thus the varying values of ρ_T) by the following formula

$$t = 1/2\ t_L + t_C$$

where t_L = loading time

$$t_C = \frac{T(H_d)^2}{c_v}$$

d. Plot ρ_T versus time.

C.3 TWO DIMENSIONAL CONSOLIDATION METHOD (Christian et al., 1972)

1. The parameters are the same as for One-Dimension Consolidation Method with the exception of the relationship between the time factor (T) and the average degree of consolidation ($\bar{U}$):

a/H_d	$\bar{U}$=10%	$\bar{U}$ =50%	$\bar{U}$=75%	$\bar{U}$=90%
1/4	.007	.09	.30	.60
1/2	.011	.14	.35	.68
1	.016	.20	.45	.80
2	.016	.24	.53	.90
∞	.008	.197	.48	.848

Where a = 1/2 B (one-half the load width or diameter).

Use linear interpolation for values of a/H_d which are not exactly listed above. When $2<a/H_d\leq 10$, use the $\bar{U}$ versus T relationship for a/H = 2. When $a/H_d>10$ use the relationship for $a/H_d = \infty$.

2. The computations are exactly the same as those of the One-Dimensional Consolidation Method.

Choice of c_v

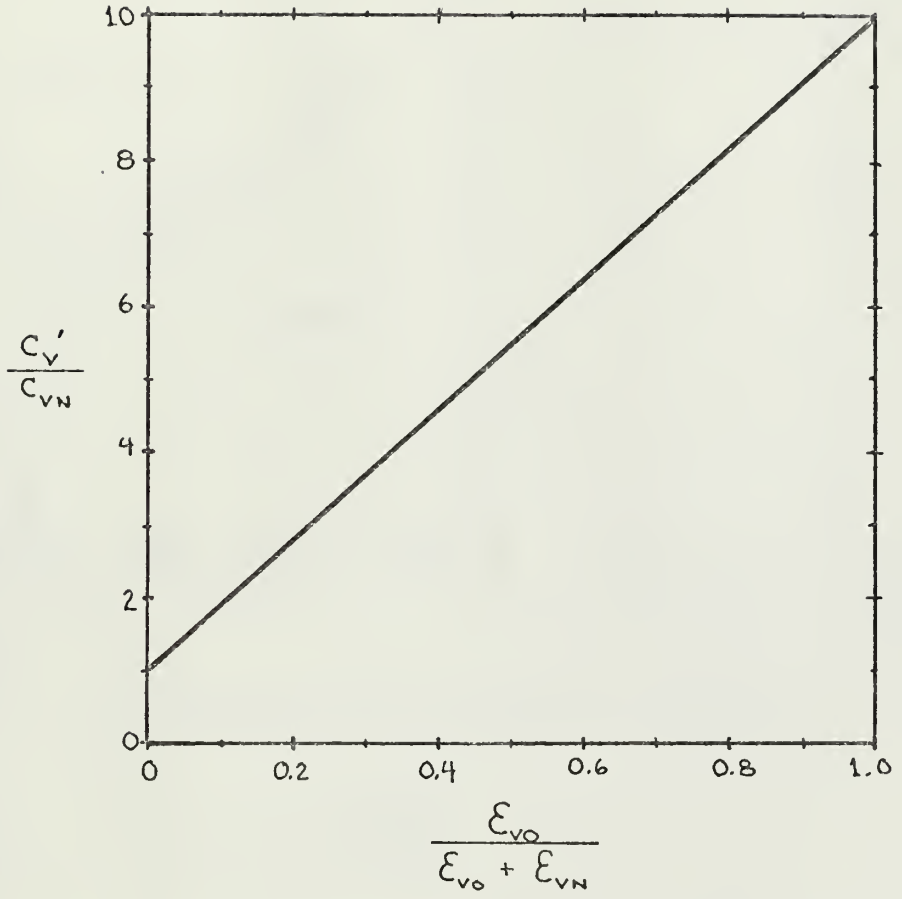
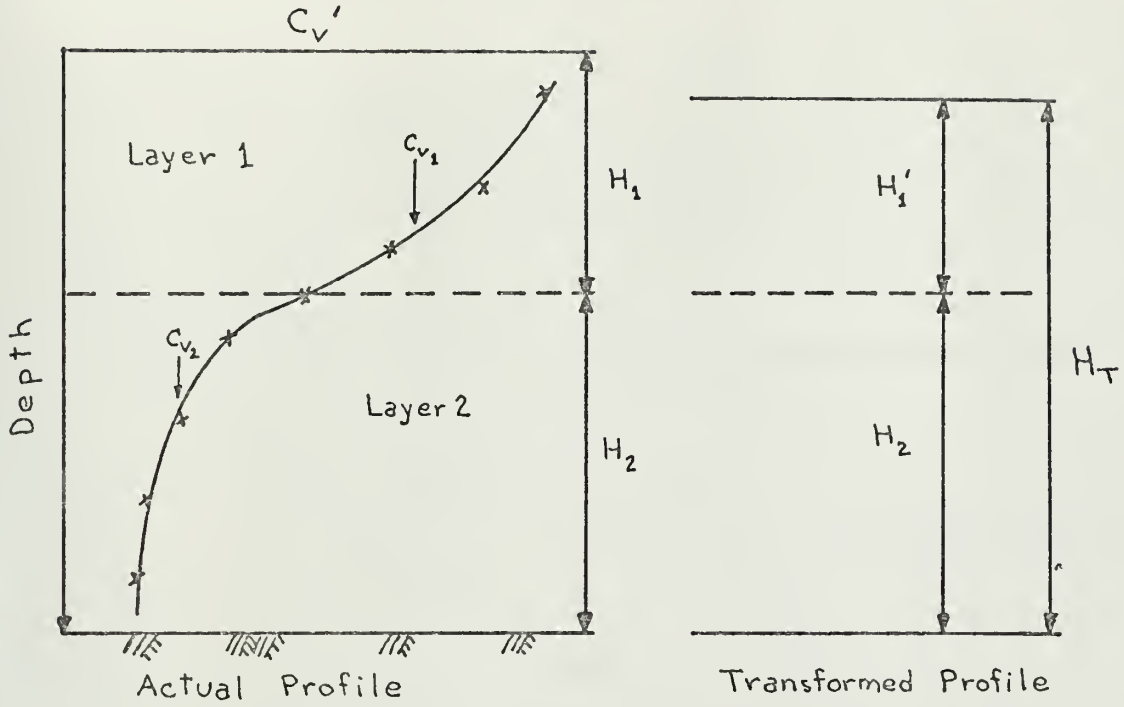


Figure C-1

Sample Transformation



Transform the thickness of Layer 1:

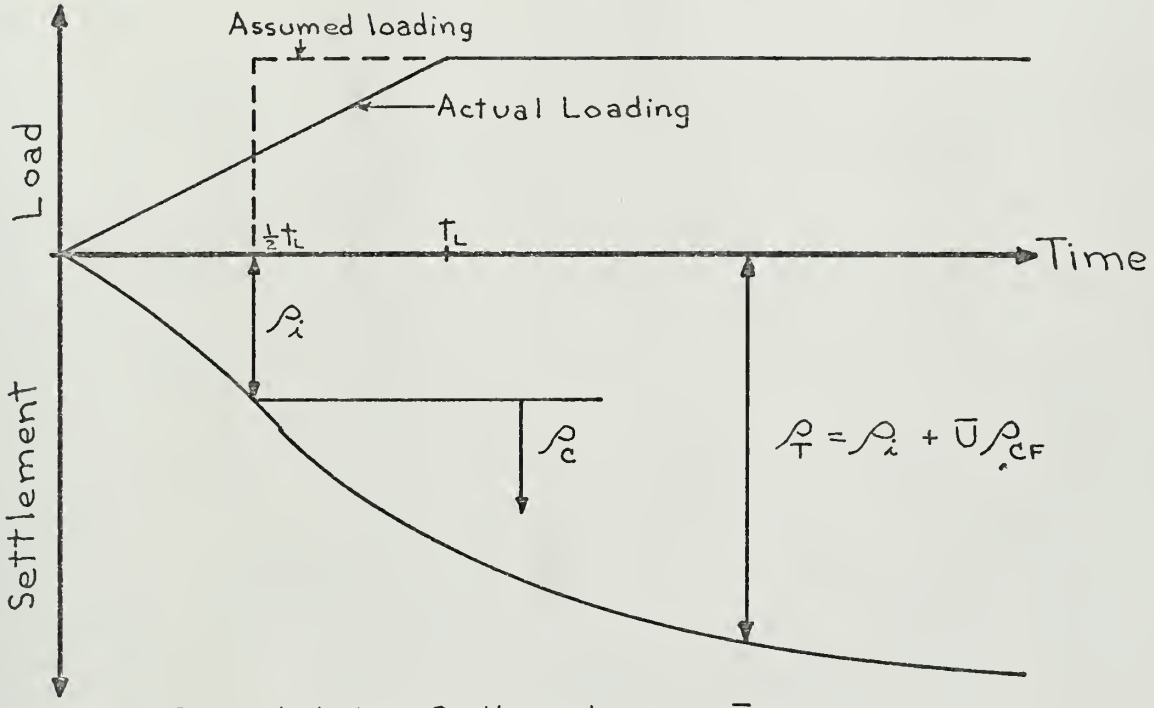
$$H_1' = H_1\sqrt{\frac{C_{v_2}}{C_{v_1}}}$$

The equivalent Total Thickness $H_T = H_1' + H_2$

The equivalent coefficient of consolidation $C_v = C_{v_2}$

Figure C-2

Sample Gradual Loading



Consolidation Settlement - $\rho_c = \bar{U}\rho_{CF}$

Initial Settlement - ρ_i

Total Settlement - $\rho_T = \rho_i + \bar{U}\rho_{CF}$

$\bar{U} = 0$ until time = $\frac{1}{2}t_L$

Figure C-3

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